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SLANTING DESIGN: A Pilot Program

FINAL REPORT

October 1985

FOR: The Federal Emergency Management Agency

Washington, DC 20472

Contract No. EMW-C-0705

Work Unit No. 1622B

Approved for Public Release

Distribution Unlimited

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American Institute of Architects Foundation

FINAL REPORT: SUMMARY

The Federal Emergency Management Agency (FEMA) under the mandate of the U.S. Congress created a pilot program to investigate the practical and cost implications of a national slanting shelter construction program. The availability of such shelters would make a critical workforce concept more viable. The intention was to incorporate shelter areas into buildings in such a manner that the affected spaces would continue to fully serve their intended functional purposes. An additional objective was to evaluate existing slanting design guidance materials, assessing their usefulness to the broad spectrum of practicing architects and engineers.

A case study method was deemed most useful for the project. The AIA Foundation (AIAF) surveyed local architectural firms and found Mariani and Associates to have two appropriate projects with building owners who were willing to entertain the possibility of shelter construction. Mariani was hired and the project begun. An office building and a hospital, both in the Washington, D.C. area were selected. Each building had a non drive-in basement with three floors of structure above. The process of design, construction documents, cost estimates, and construction were observed. Models of the hospital shelter were tested for structural integrity.

The office building was located in downtown Washington at 727 15th Street, N.W., just two blocks from the White House and the Treasury Building. The project involved the demolition of a single story movie theater and the restoration and incorporation of its historic facade into a new eleven-story office structure.

The original hospital project was located at 13th and V Streets, N.W. on a 2.4 acre full block site. Parts of an existing complex were to have been renovated and parts demolished to allow for new construction. The two projects represented a wide cross-section of the types of problems and impediments which might be encountered in a national scale construction program.

The design of the shelter in the office building was straightforward because the program and plan were simple. Major problems were of three types; structural, financial and approval related. The structural difficulty involved the fact that the added dimension required for the floor and ceiling of the shelter necessitated taking the shoring and footings below those of the two adjacent buildings. The additional costs associated with this were not directly related to the construction of the shelter and eventually led to the decision not to construct. The financial problems were related to the sluggish economic climate which existed. The speculative developers proceeded quite slowly hoping to acquire more favorable loan monies. The approval problems related to the fact that the project had to be submitted to both the Fine Arts and Landmarks Commissions several times, with each submittal extending the design process.

Four alternative designs were considered for the hospital located at 13th and V Streets. A design was selected and detailed, construction documents were prepared and submitted to federal financing agencies only to lead to a complete change of site and total redesign of the project. On the new site the design proceeded simply, the shelter was located in an auditorium below the main entrance tc the hospital. Detailed design, structural calculations, construction bids, and contractor selection were all completed. Problems and delays were caused by a change of ownership of the hospital, a labor strike, and a legal dispute related to an adjacent parking structure. Construction difficulties were minimal. The blast shelter added a new layer of complexity which necessitated additional planning and scheduling but no extraordinary constraints.

Construction cost estimates for the two projects ranged between $37 and $110 per square foot. No definitive explanation for this wide range can be offered.

The design guidance materials used in this project were found to provide accurate information but to be inadequate for use in the type of national construction program envisioned with the critical workforce concept. "Protective Construction", TR-20, (Vol.4) and the other documents were judged to be more textbooks than design manuals; requiring reference to other documents to clarify definitions, terminology, and symbols. The ideal design manual should present straightforward examples, problems and many charts and tables to assist the busy, and probably inexperienced designer.

Two models (at one-fifth scale) of the hospital shelter were constructed and tested. One model was exposed to 15 psi over-pressure, as designed. This model suffered no damage. The second model was exposed to 50 psi overpressure and suffered only minor damage in the form of hairline cracking.

Conclusions are threefold:

- A national shelter construction program is thought to be feasible, but precise projecr scheduling woulc be extremely difficult.

- Existing design guidance is thougth to be inadequaie. No national construction program should be contemplated prior to the preparation of simplified, straightforward design manuals.

- No firm assessments of the costs of a national construction program can be made.

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The AIA Foundation contracted with Mariana and Associates to design basement blast shelter areas in two buildings in Washington, DC. Alternate construction bids were received for both designs. FEMA decided to finance shelter construction at the National Rehabilitation Hospital (NRH) located at 106 Irving Street NW, and not to finance shelter construction at 727 15th Street NW. Incremental shelter construction cost estimates ranged between $37 and $110 per square foot above normal costs.

(see reverse side)

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Evaluation were performed for two questions: (1) the adequacy of design guidance provided to practicing architects and engineers by "Protective Construction", TR-20 (Vol.4), and other existing references -- Found to be inadequate; and (2) the practical difficulties of incorporating shelter construction into otherwise typical building projects -- Found to be manageable, but difficult to schedule precisely.

The shelters were designed to sustain 15 psi (pounds per square inch) overpressure. Fifth scale models of the NRH shelter were tested at White Sands, NM in July 1985 at overpressures of 15 psi and 50 psi. The model tested at 50 psi sustained only very minor damage in the form of hairline cracks. The model tested at 15 psi sustained no apparent damage.

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FINAL REPORT: SLANTING SHELTER DESIGN

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ABSTRACT

The AIA Foundation contracted with Mariani and Associates to design basement blast shelter areas in two buildings in Washington, D.C. Alternate construction bids were received for both designs. FEMA decided to finance shelter construction at the National Rehabilitation Hospital (NRH) located at 106 Irving Street, N.W. and not to finance shelter construction at 727 15th Street, N.W. Incremental shelter construction cost estimates ranged between $37 and $110 par square foot above normal costs.

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OBJECTIVES

The United States Congress mandated in the 1981 Defense Appropriations Act that the Federal Emergency Management Agency (FEMA) create a pilot program to investigate the practical and cost implications of a national shelter construction program. The pilot program was to consist of designing and constructing a minimum of two buildings with an enhanced ability to withstand nuclear explosions while sustaining minimized damage. The program was an approach to maximizing nuclear effects protection in risk areas in the United States and was to provide shelters which are strong enough to survive high overpressure (above 15 psi). The availability of such shelters would make a critical work force concept more viable. The major objective of the effort was to measure the additional costs and to assess the practical difficulties associated with the design and construction of buildings that incorporate current slanting design and blast resistance guidance. The intention was to incorporate shelter areas into buildings in such a manner that the affected spaces would continue to be fully functional still serving their original purposes.

An additional objective was to evaluate the efficacy and usefulness of existing slanting shelter design guidance for such a national shelter construction program as would be required to sustain the critical workforce concept.

METHODS

A case study method was deemed most useful.

1. The shelters were to be designed and constructed in buildings which did not have drive-in basements.

2. The AIA Foundation (AIAF) searched for and identified two commercial buildings (a hospital and an office) in the Washington, D.C. area for which basements were planned.

3. The AIAF contracted with Mariani and Associates who were the architects of both buildings.

4. Mariani obtained the approval of the building owners for the inclusion of the alternative shelter designs in the two projects.

5. Several alternative shelter locations were selected and presented to FEMA.

6. Final shelter locations were selected and designs created.

7. Engineering calculations and construction documents for the shelter areas were prepared.

8. Construction bids were requested for the shelters as alternates to the base bids for the building construction.

9. Yes/No construction decisions were made for both shelters.

10. Construction of the hospital shelter was begun and completed.

11. Waterways Experiment Station under a separate contract to FEMA constructed two fifth-scale models of the hospital shelter. One model was blast-tested at 15 psi overpressure; the other at 50 psi.

DISCUSSION OF RESEARCH:

Selection of Architects:

At the outset of the project, October 1981, the AIA Foundation performed a survey of architectural firms in the Washington, D.C. metropolitan area to find what types of projects they had underway. AIAF was looking for commercial buildings planned to have a minimum of three stories above grade. The buildings were to have basements which were not of the drive-in type. The building projects were required to be at a preliminary design stage so that incorporation of alternate slanting shelter design would not disrupt the schedule or increase costs to the building owner or developer.

The firm of Mariani and Associates, Inc., located at 1600 20th Street, N.W. was found to have two such projects at that time. The firm also had considerable shelter design expertise from previous work and was judged to be an appropriate subcontractor to work on the project. Mariani requested and received tentative approval from its clients to use the two buildings as the case studies in this project. An AIAF subcontract was subsequently written with Mariani.

Selection of Building Sites:

The two selected building projects were a speculative office building and an addition to a hospital. The office building was located in downtown Washington at 727 15th Street, N.W., just two blocks from the White House and the Treasury building. The project involved the demolition of a single story movie theater and the restoration and incorporation of its historic facade into a new eleven-story office building with basement level below (see Figure 1). The building was planned to have a reinforced concrete structure and contain a total of 50,500 square feet of office and ground floor commercial space. The basement area was to be 4,500 square feet and because no parking was planned, more than one underground level could have been designed as shelter area should that have proven desirable.

The hospital project, the National Rehabilitation Hospital, was located at 13th and V Streets, N.W. on a 2.4 acre full block site. Of the existing building complex, the structures on the northern portion of the site were to be renovated, while those on the southern portion were to be razed to make room for new construction. The renovated segment included basement-level receiving, supply, and laboratory areas and a 5,000 square foot medical records storage area which could have accomodated a shelter if that had been desired. The renovated floors above were to contain administrative, diagnostic and treatment facilities. Both the new and renovated parts of the building were to be column-supported concrete slab construction with exterior

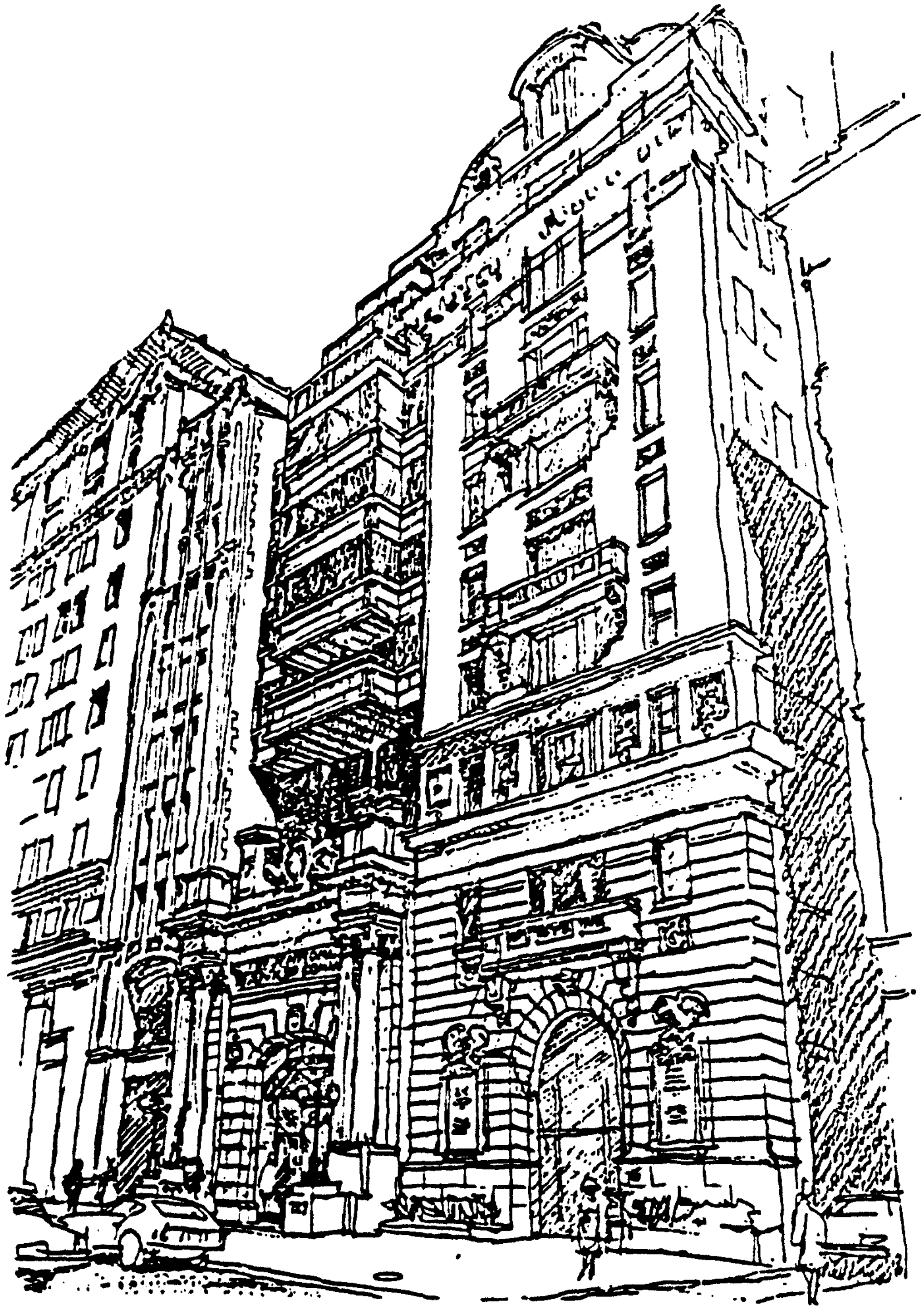


FIGURE 1 -- Architect's Sketch of Office Facade -- 727 15th Street

masonry cavity walls. The total area proposed was 380,000 square feet of which 267,000 was to be new and 113,000 renovated.
Both of the above buildings had basement areas which would accomodate blast-resistant design as well as their intended conventional functions. In addition, the two projects represented a wide cross-section of the types of problems and impediments which might be encountered in a national scale shelter construction program trying to incorporate shelter design into otherwise typical building projects.

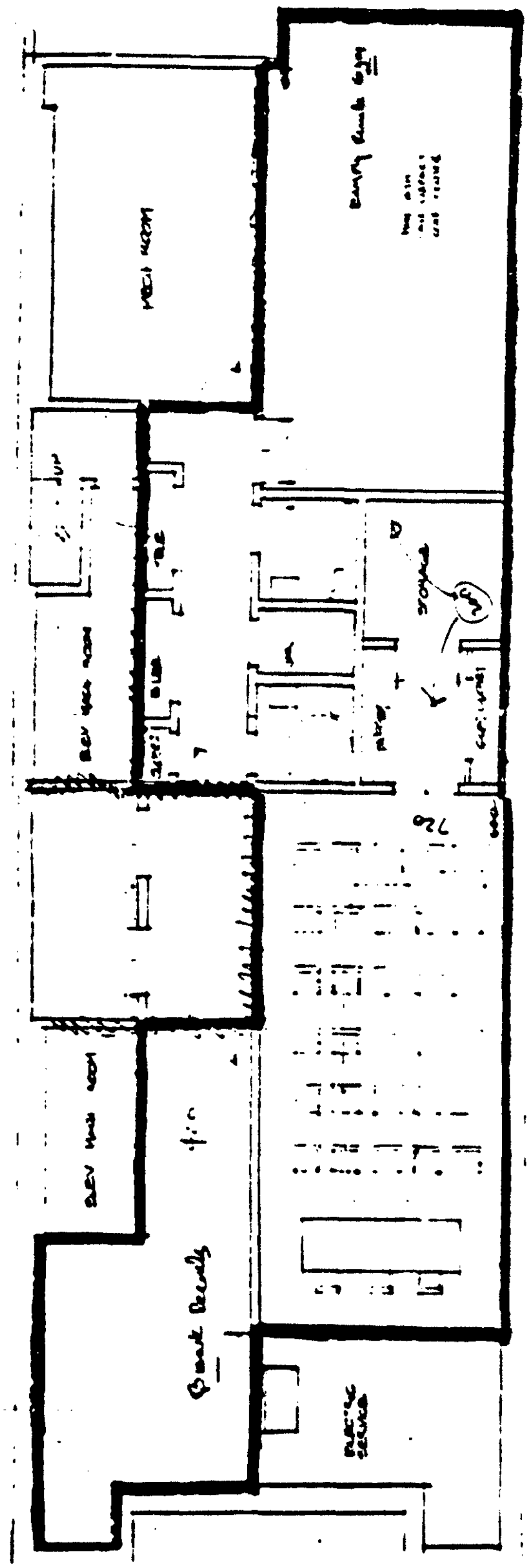


FIGURE 2 -- Preliminary Plan of Office Basement with Shelter

Design Process: Bank/Office Building (727 15th Street)

Design Alternatives. The design of the blast shelter was begun in October of 1982. Preliminary design of this blast shelter was quite straightforward. The overall basement plan was simple and there were only a few options to consider. The major consideration was the exact location of the blast resistant wall relative to the elevator and stair tower (see Figure 2). The owner's program for the basement changed several times. For example, one corner went from storage area to a health club/gym. This caused adjustments to the area of the blast shelter. Fortunately, the gym was given up and the space became storage again.

A problem which was a concern for both aesthetic and structural reasons had to do with the ceiling height dimension in the conference/board room. The floor to ceiling heights for the building were tight to begin with and adding several inches for shelter construction made the problem worse. The conference space was a large area which needed a higher ceiling height in order to avoid a cramped feeling. Dropping the floor level was not desirable because that put the foundations below those of the adjacent buildings. This lowered floor level would cause shoring, structural, and construction problems which would lead to additional costs beyond those directly related to the shelter.

Detailed Design/Construction Documents. By February of 1982 three months delay had been experienced in the design of this project. These delays were caused because the architects had to go through the approval processes of the Fine Arts Commission and the Landmarks Commission.

Serious delays were experienced during the period ending in June of 1982. The architects were kept on hold while waiting for decisions from the owner. The project was speculative and the high interest rates and uncertainty in the financial markets provided a strong incentive for the owner to be deliberate in such decisions as the type of heating, ventilation and air-conditioning system to be installed.

Construction documents were begun in August of 1982, but again the owners were not rushing to complete the project. The construction documents were 90 percent complete at the end of February, 1983. The drawings were finished by the end of May.

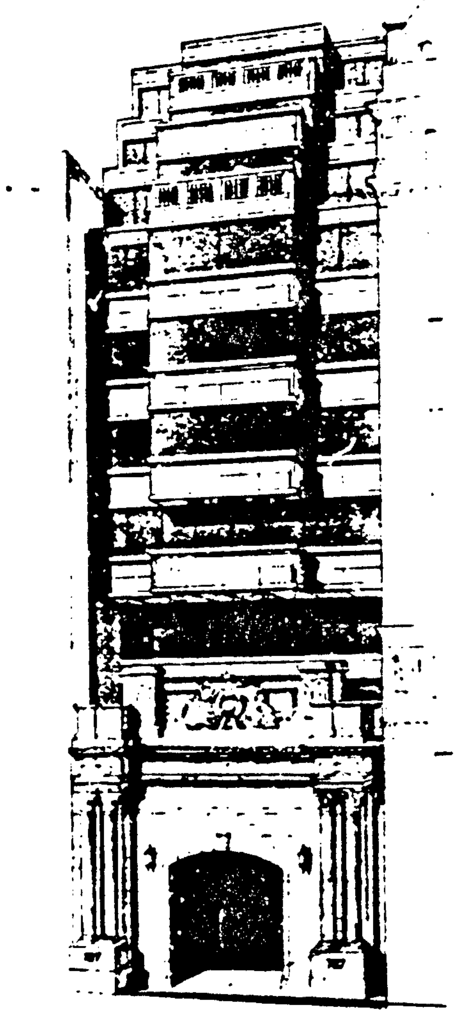
Bid Documents. Construction bids were requested for the project in September of 1983. A construction management company directed the bid process, receiving piece bids from subcontractors for various segments of construction such as steel, concrete and glazings. When recieved, the bids for general construction were 50 percent over the developer's budget. This serious problem again stopped the entire process. Alternate bids for the blast shelter and other alternates were not completed at this point. The major problems were the cost of both the steel structure and the marble facade. The redesign of the facade involved resubmission to the Fine Arts Commission and additional time delays.

The owners of the development project decided at this juncture for financial reasons that work had to proceed rapidly. In November of 1983 during the redesign process it was determined that construction of the blast shelter would definitely require taking the overall building foundation below those of the adjacent buildings. It was also determined that without shelter construction this would not be necessary. Based on this information an alternate construction bid was produced for the shelter area. The construction estimate of $175,000 calculated to be $110 per square foot for the 1,590 square feet designated for the shelter.

At that point FEMA decided not to construct the blast shelter in the office building. The decision was based on the fact that 43 percent of the estimated construction costs were not directly associated with the blast shelter itself. Of the total costs $75,000, or $47 per square foot, was required for additional underpinning necessitated by the existence of the blast shelter but not its design. The added depth of both the ceiling and the floor of the shelter required that the building foundation be two feet lower than it would otherwise have been. This added expense was created because the increased dimension put the new foundation below those of the two adjacent structures.

Conclusions. This project (see Figure 3) demonstrated a wide variety of problems which can be encountered in the construction industry. One of the most amusing was that a preservation group called "Don't Tear It Down" opposed the demolition of the theater based on the claim that the open air above the building was an "historic open space". The mixture of factors involved clearly demonstrates that the inclusion of government sponsored shelter areas in a variety of building construction should be expected to take considerably more time than the ideal fast-track scenario might indicate. This particular building witnessed speculative development occurring during a period of economic recession and escalating construction costs. Time was devoted to obtaining reviews and approval from a variety of boards and agencies. In addition to the preservation group and the Fine Arts and Landmark Commissions already mentioned, the project had to be reviewed by the White House security police because of its proximity to 1600 Pennsylvania Avenue. The prime objective of this project, to observe the integration of shelter design into the exigencies of a specific ongoing architectural project, was well met by this building.

The Presidential Point Of View



727 15th Street, NW

Just steps from the White House, 727 15th Street is the epitome of Capital city prestige. This historic address has been fully updated—a distinguished blend of the past and present—with full floor office suites of 3600 sq. ft., dramatic balcony views, state-of-the-art security and office interiors finished to your exacting specifications.

A development of
First Washington
Development Group, Inc.

Leasing by
First Capital
Realty, Inc.
232-4220

FIGURE 3 -- Developer's Advertisement with Completed Facade -- 727 15th Street

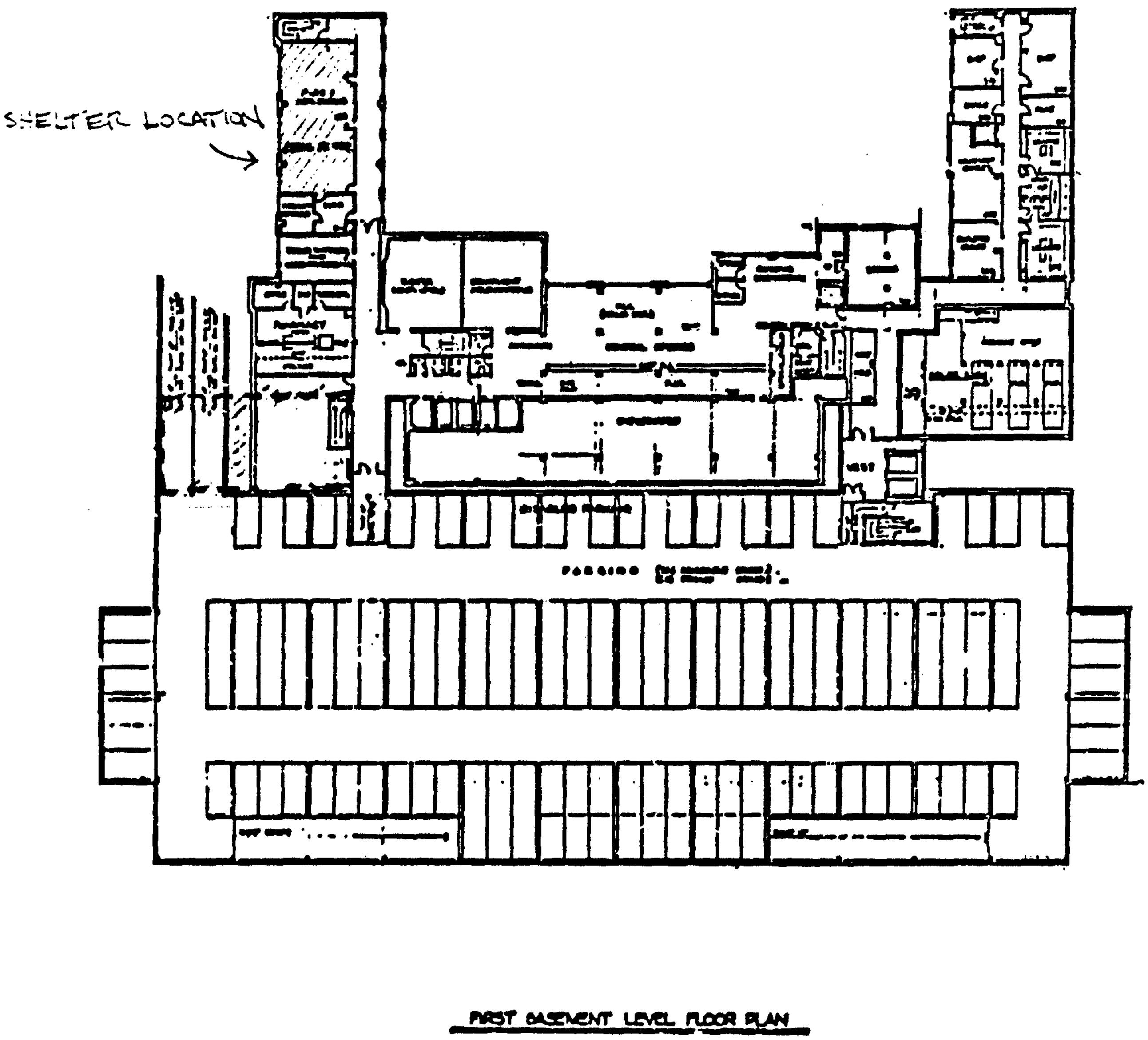


FIGURE 4 -- Hospital Alternative #1 -- First Basement Plan

Design Process: National Rehabilitation Hospital

Design Alternatives. The original design concept involved placing the blast shelter in the medical records area of the existing hospital facility (see Figures 4). Questions arose when AIAF discovered that this section of the hospital was to be partially exposed above grade. Discussion with FEMA led to the determination that renovation type construction was not desired.

A major design objective within the project was to incorporate the blast shelter into an area of the building where the space would still be fully functional on its own right. It was intended that the design not entail any significant structural or construction alterations. The incremental construction costs were to be restricted as much as possible to those for additional material and not for altered design.

The reason stated above led to the rejection of a second alternative location for the blast shelter. That design alternative was the placement of the shelter beneath the lowest parking level in the new construction area (see Figures 5-8). This proposed space was not to be included on the non-shelter construction documents which meant that the incremental costs would have included the entire construction costs as well as additional expenses for excavation and shoring.

Another location was considered and rejected (see Figure 9). The theater on the existing first floor (north side) had a large

FIGURE 5 -- Hosrital Alternative #2 -- Shelter Plan

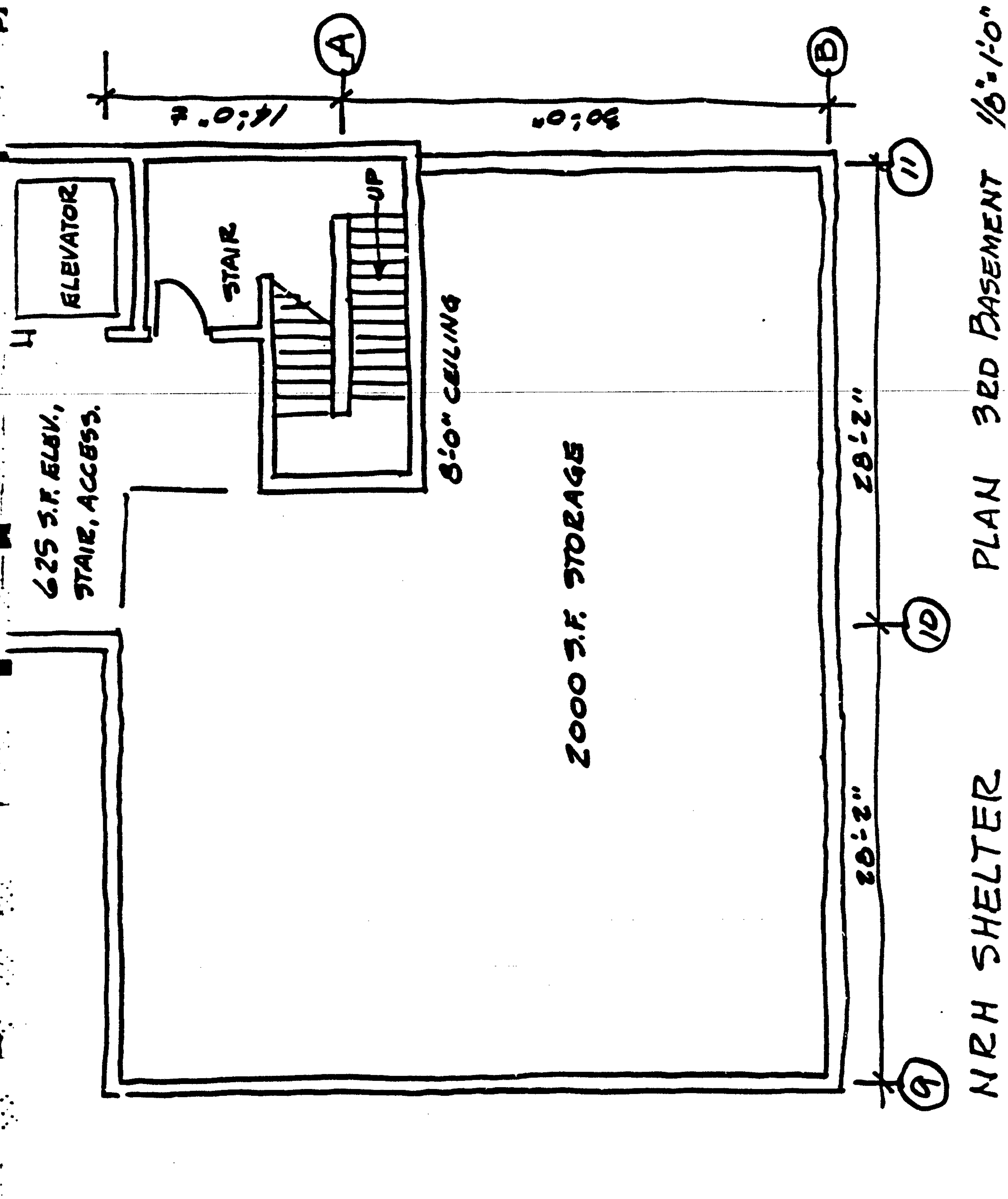


FIGURE 6 -- Hospital Alternative #2 -- Sections & Elevation

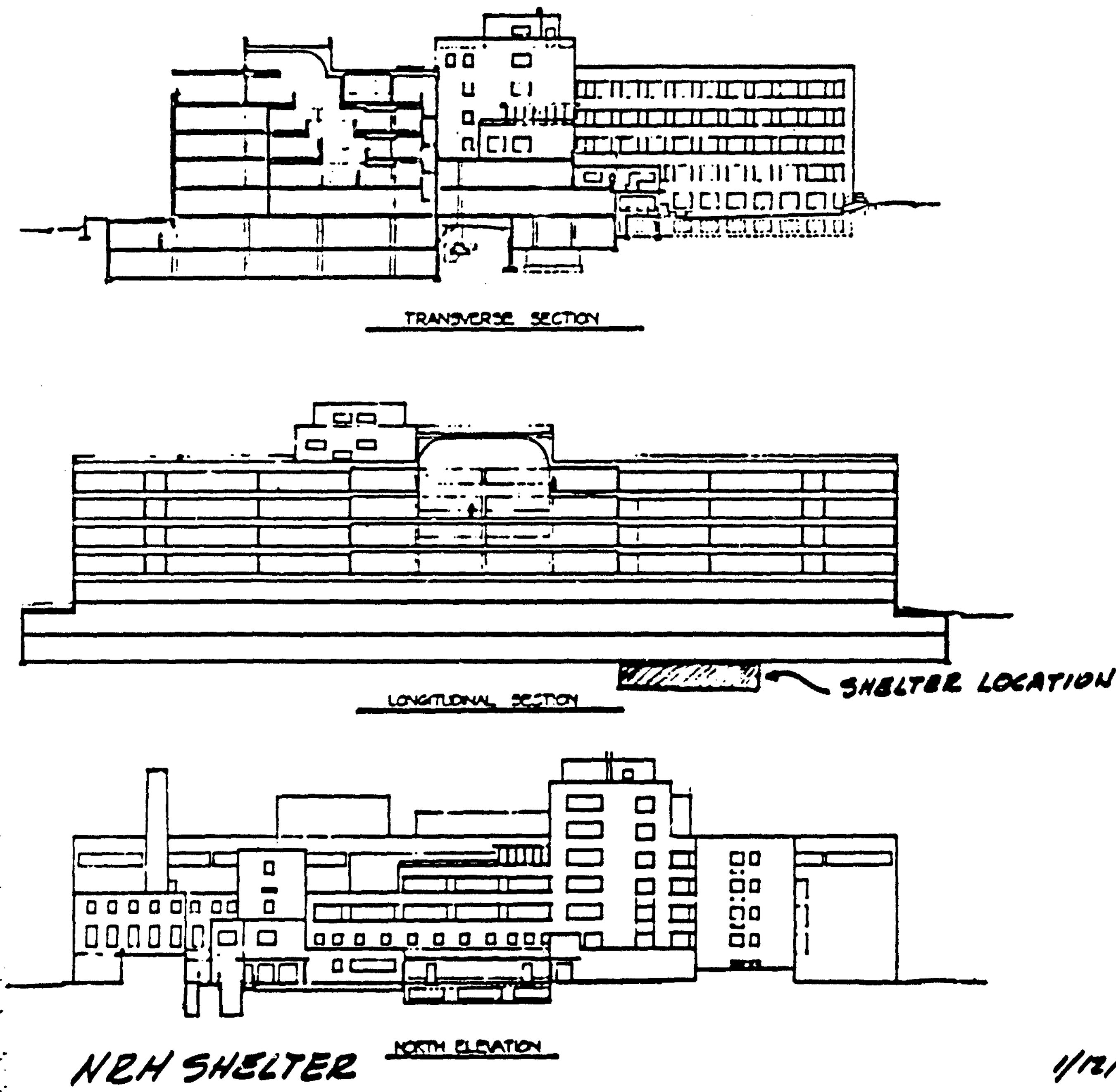
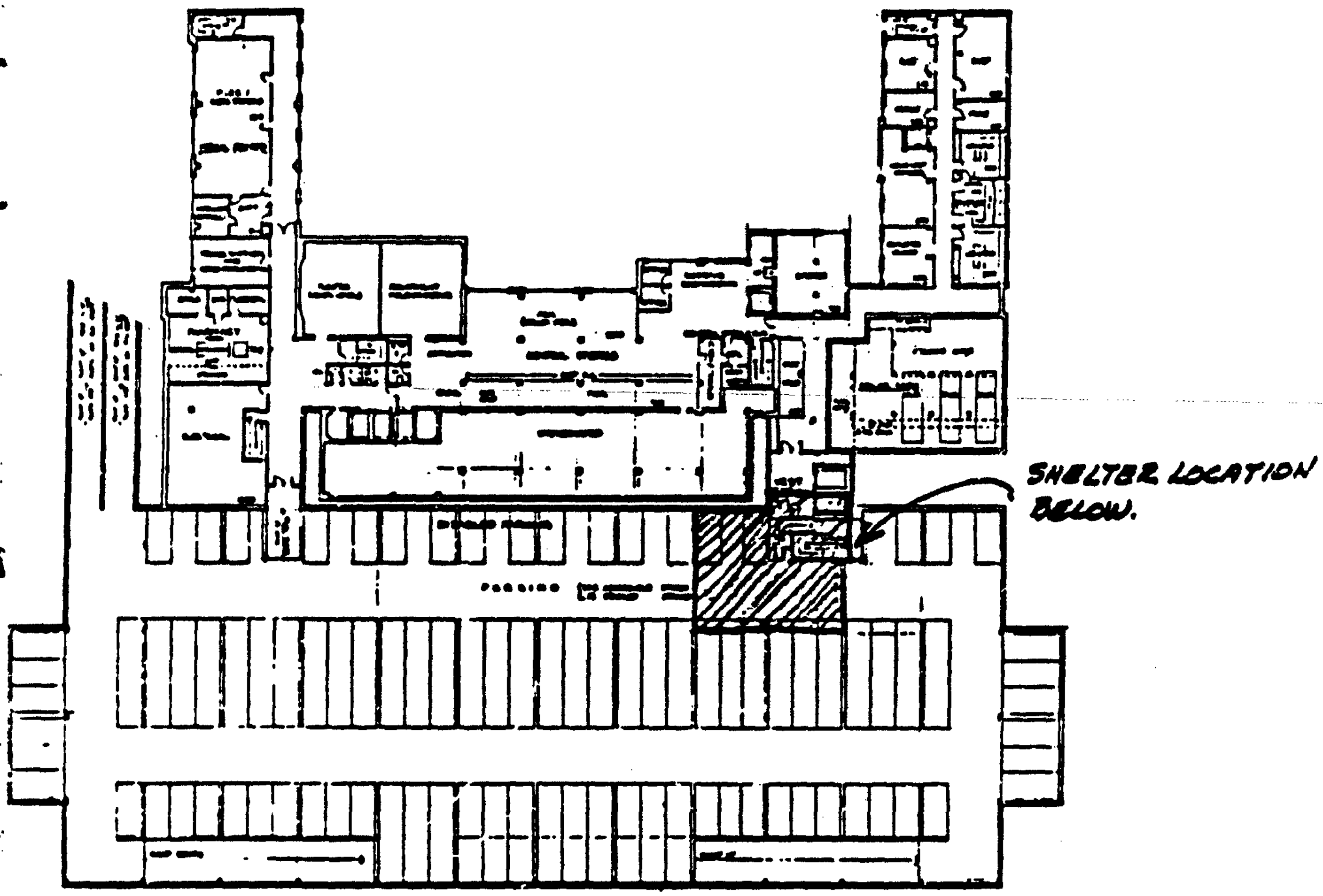


FIGURE 7 -- Hospital Alternative #2 -- First Basement Plan



FIRST BASEMENT LEVEL FLOOR PLAN C

NRH SHELTER

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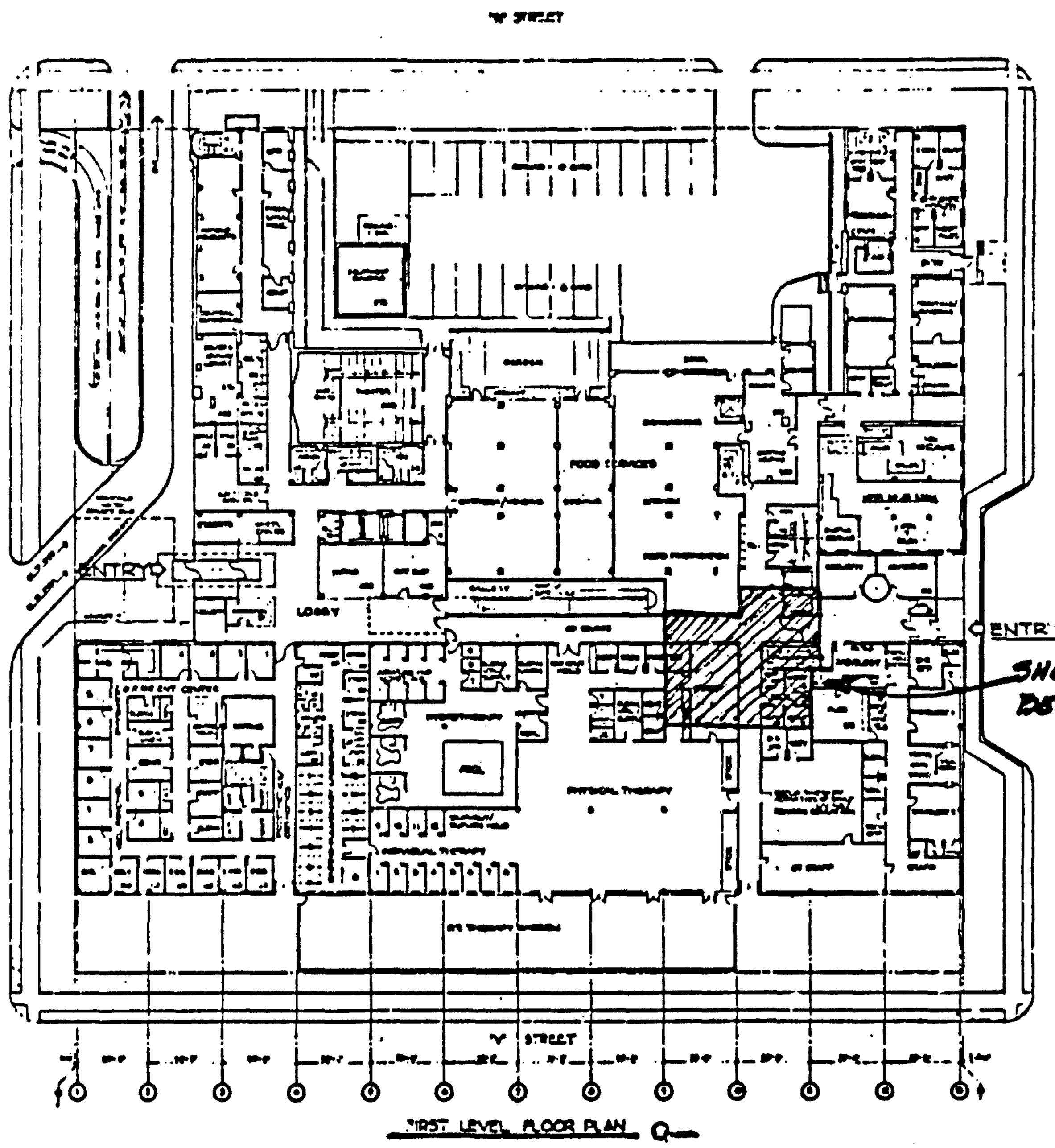
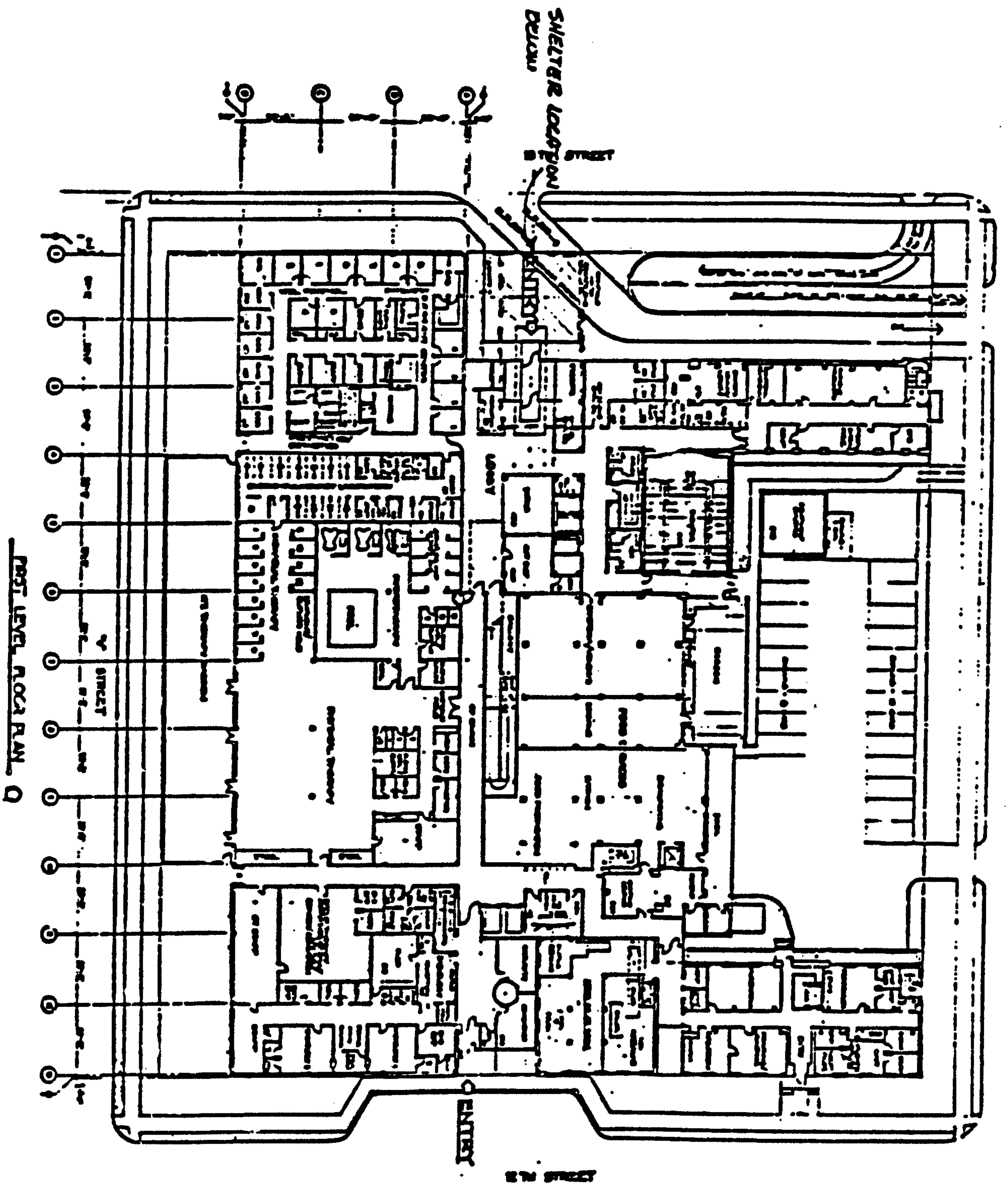
FIGURE 3 -- Hospital Alternative #2 -- Second Basement Plan



NRH SHELTER

1/12/

FIGURE 9 -- Hospital Alternative #3 -- First Floor Plan



un-utilized basement which was to be renovated. This alternative did not work for several of the reasons enumerated previously. It would have involved renovation, and significant structural modifications would have been required.

The last concept was placement of the blast shelter on the grade of the lowest parking level (see Figure 10-13). One potential problem with that location was that it fell immediately below the garage ramps. This location was selected and detailed design commenced in March of 1982.

Detailed Design/Construction Documents. In June of 1982 changes to the facade design of the hospital required redesign of the structural column grid. This problem caused delays for the entire project as well as the shelter design.

By August 1982 substantial progress has been made on production of the construction documents. The hospital project was eligible for federal loan monies from the U.S. Department of Housing and Urban Development. Mariani and Associates and their engineering consultants were required to prepare and submit not-to-exceed cost estimates to the HUD program. HUD also required structural and other revisions. Final blast shelter design and analysis had been scheduled for completion by late September in coincidence with the request for construction bids. After submittal of two sets of construction documents to the HUD funding agencies, the decision was made to completely change the building site.

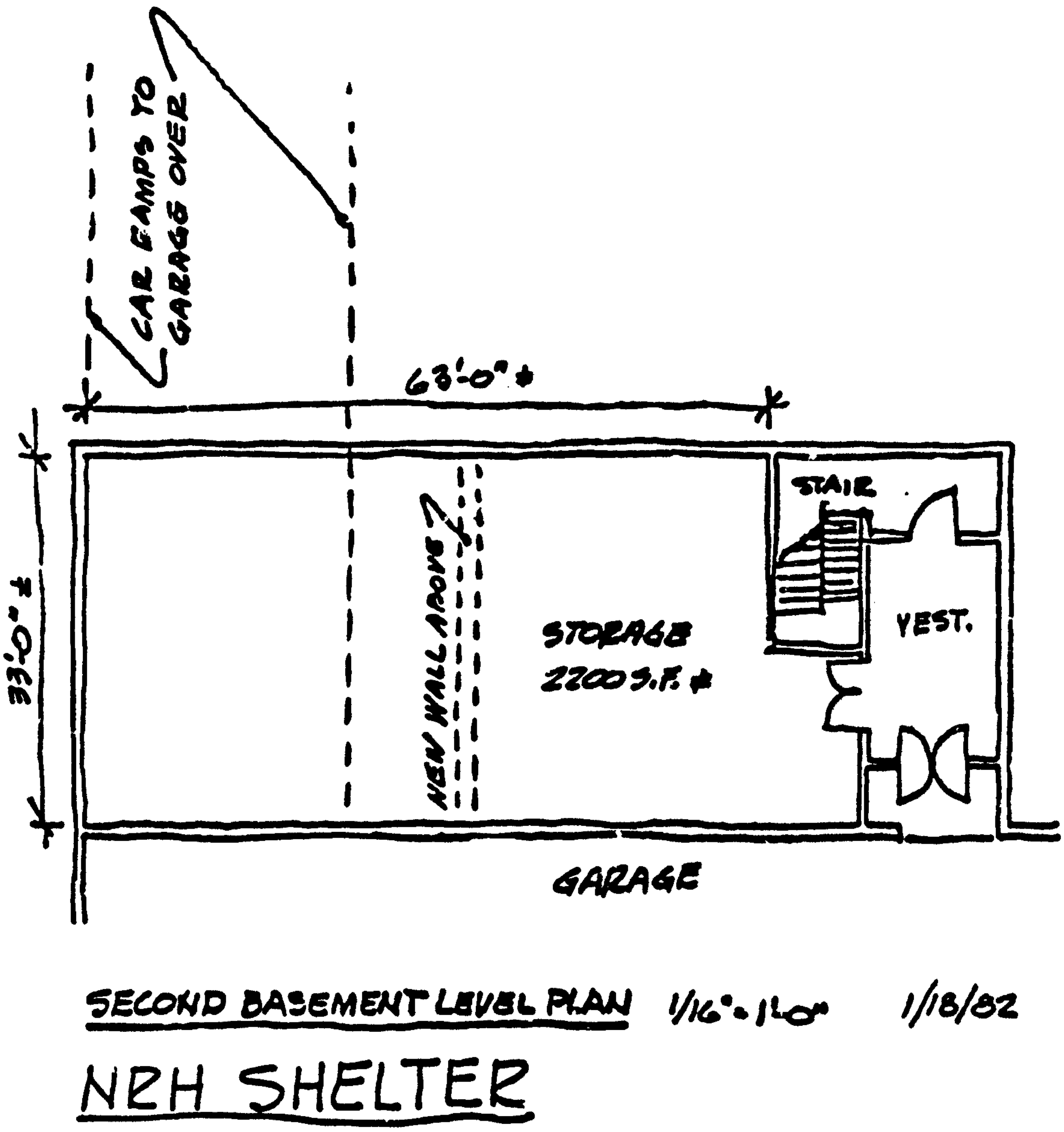
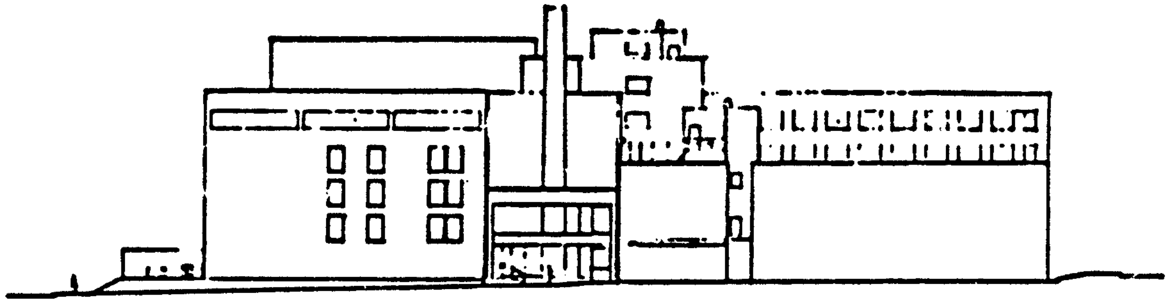
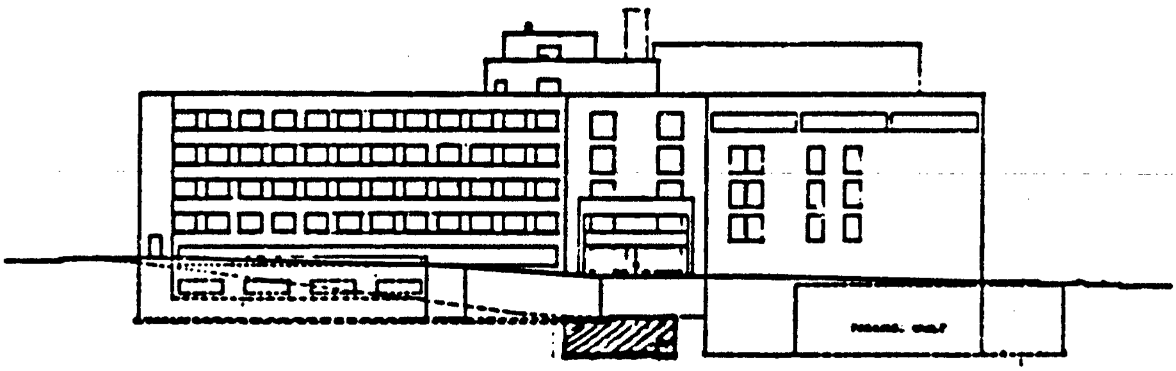


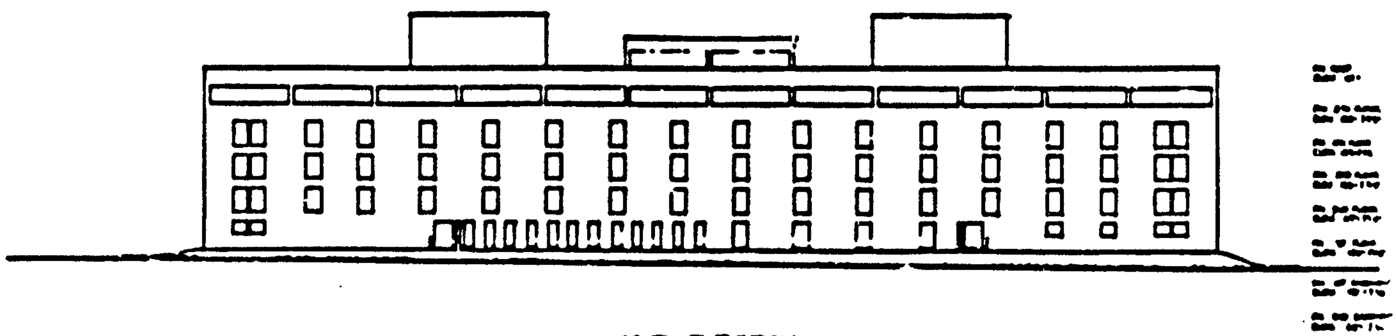
FIGURE 10 -- Hospital Alternative #4 -- Shelter Plan



EAST ELEVATION



WEST ELEVATION



SOUTH ELEVATION

FIGURE 11 -- Hospital Alternative #4 -- Sections & Elevation

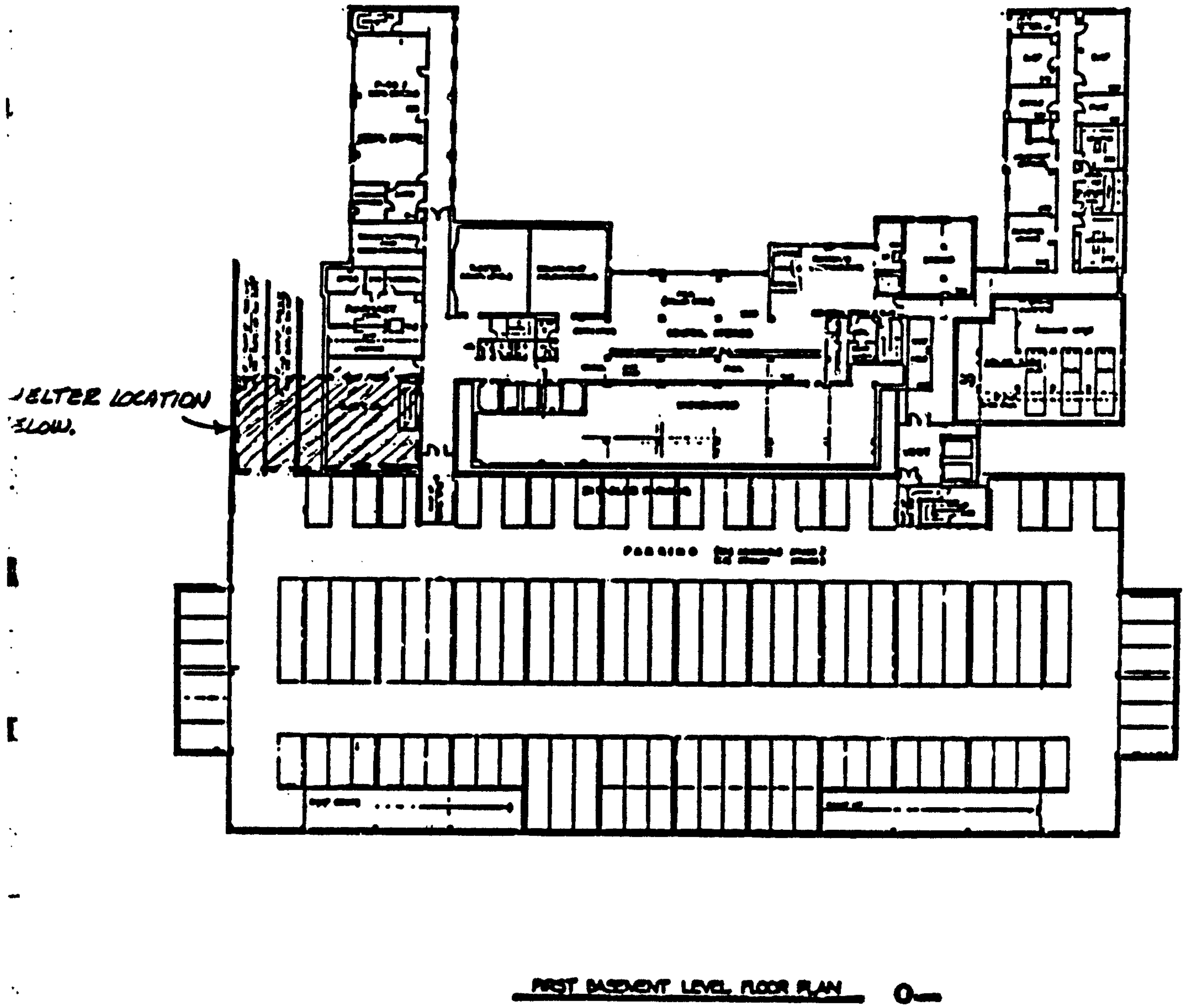
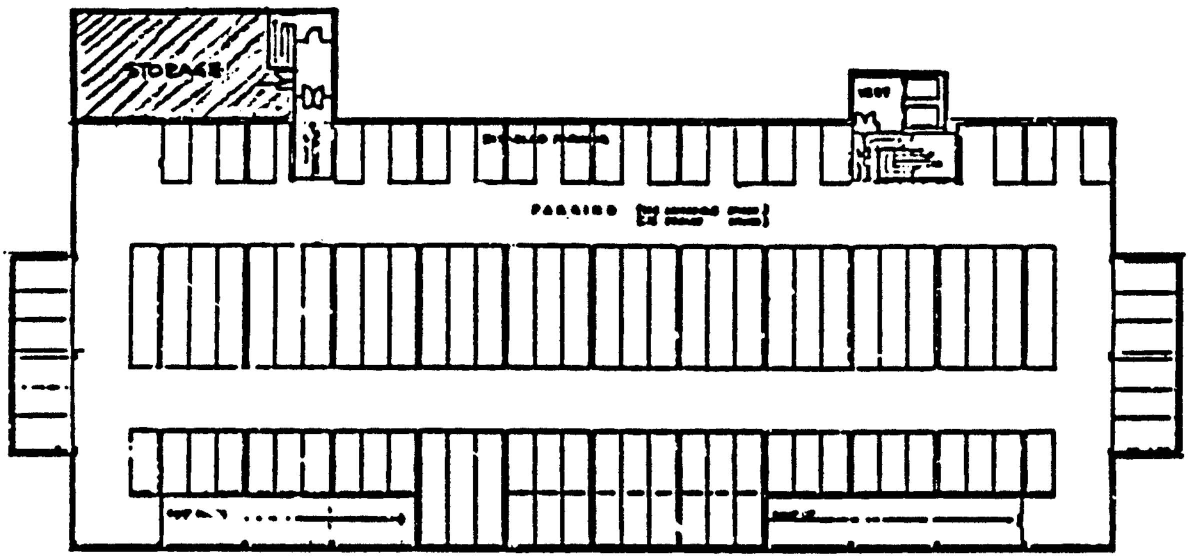


FIGURE 12 -- Hospital Alternative #4 -- First Basement Plan



SECOND BASEMENT LEVEL FLOOR PLAN

FIGURE 13 -- Hospital Alternative #4 -- Second Basement Plan

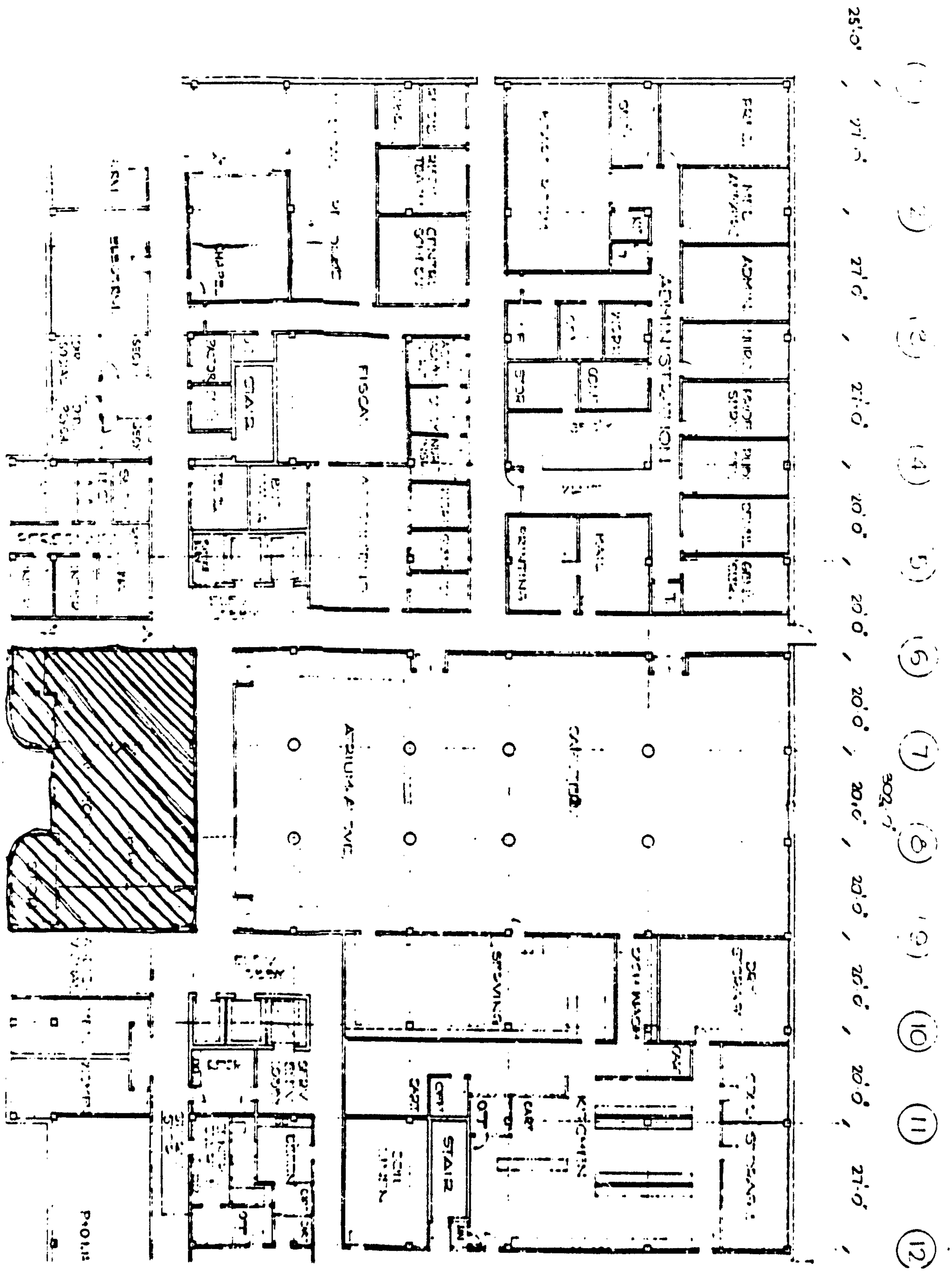
The new location was on the Washington Hospital Center site adjacent to Children's Hospital at 106 Irving Street, N.W. This decision necessitated a complete redesign of the hospital and took this shelter design back to the initial stage of selecting alternative locations.

By November of 1982 the shelter location was selected (see Figure 14), alternative designs were considered, and structural design calculations (see Figures 16-20) were completed. These preliminary calculations, by Don Neubauer, PE, are included as Appendix A of this report. A preliminary construction bid from Turner Construction Company was received at this time (see Figure 15).

During August of 1983 detailed design and construction documents were completed.

Bid Documents. Construction bids were received from two companies; Turner Construction (see Figure 21) and George Hyman Construction (see Figure 22). Turner's bid for the blast shelter alternate was \$140,000 while Hyman's was \$87,000. Dividing by the 2,300 square feet of shelter area these bids translate to \$60.87 per square foot for Turner and \$37.83 for Hyman. No satisfactory explanation is available for the large discrepancy in construction bids. Unfortunately, FEMA did not have influence on the ultimate selection of the construction company. FEMA's

FIGURE 14 -- Hospital Alternative #5 (FINAL) -- First Floor Plan



irner

Turner Construction Company
Pennsylvania Avenue N.W.
Washington D C 20004
Telephone (202) 393 5100

RECEIVED NOV 12 1982

November 12, 1982

Mr. T.F. Mariani
Mariani & Associates
1600 20th St., NW
Washington, D.C. 20009

Subject: NRH Blast Resistant Shelter Study

Dear Mr. Mariani:

We have completed our cost study for the addition of a Blast Resistant Shelter at the ground floor auditorium of the National Rehabilitation Hospital. We value the additional work at $ 120,000 making the new Parameter Estimate total $ 19,095,000. (See attached summary sheet).

The study was based on Untitled Drawings prepared by your office dated 11/9/82. The unusually large size beams, walls, columns, and slab coupled with dense reinforcing steel accounted for the relatively high unit cost for the 2,2[illegible] sq. ft. shelter.

If you have further question please do not hesitate to call me.

Very truly yours,

Thomas J. Paci
Chief Estimator

PARAMETER ESTIMATE
National Rehabilitation Hospital
at the Childrens' Hospital Campus

Excavation & Foundations		$ 899,000
Structural Frame		3,995,000
Roofing & Waterproofing		459,000
Exterior Wall		1,498,000
Interior Finishes		2,481,000
Special Requirements		381,000
Vertical Transportation		410,000
Plumbing		1,107,000
Fire Protection		402,000
HVAC		2,179,000
Electrical		2,047,000
Site Work		544,000
DIRECT COST		16,402,000
General Conditions @ 8%		1,312,000
SUB TOTAL		17,714,000
Contingency @ 3%		531,000
SUB TOTAL		18,245,000
FEE @ 4%		730,000
TOTAL		18,975,000
BLAST RESISTANT SHELTER	$ 115,000	
ASSOCIATED FEE @ 4%	5,000	
		120,000
		$ 19,095,000

FIGURE 15 -- Preliminary Construction Bid -- Turner

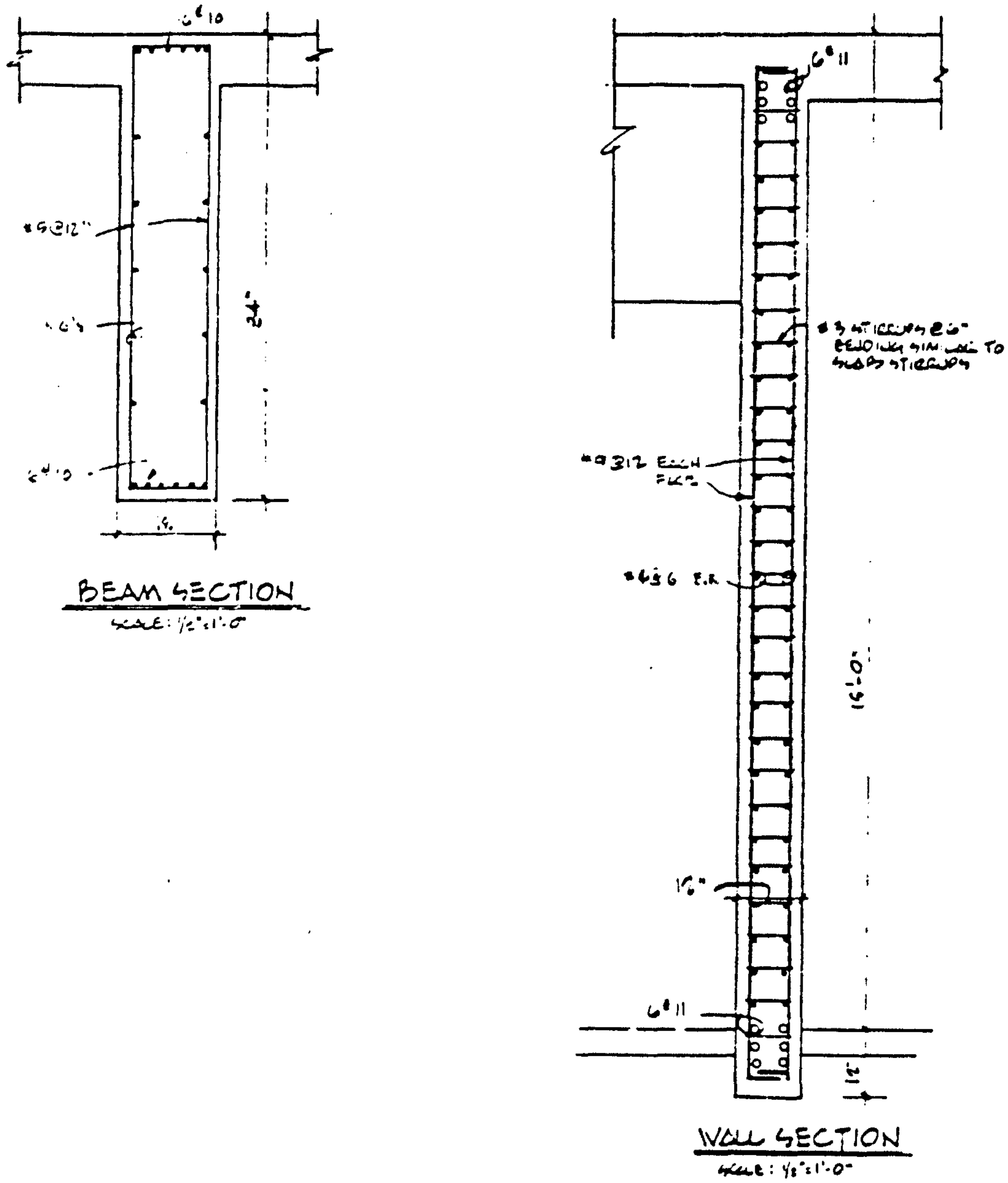


FIGURE 16 -- Structural Details -- Sections

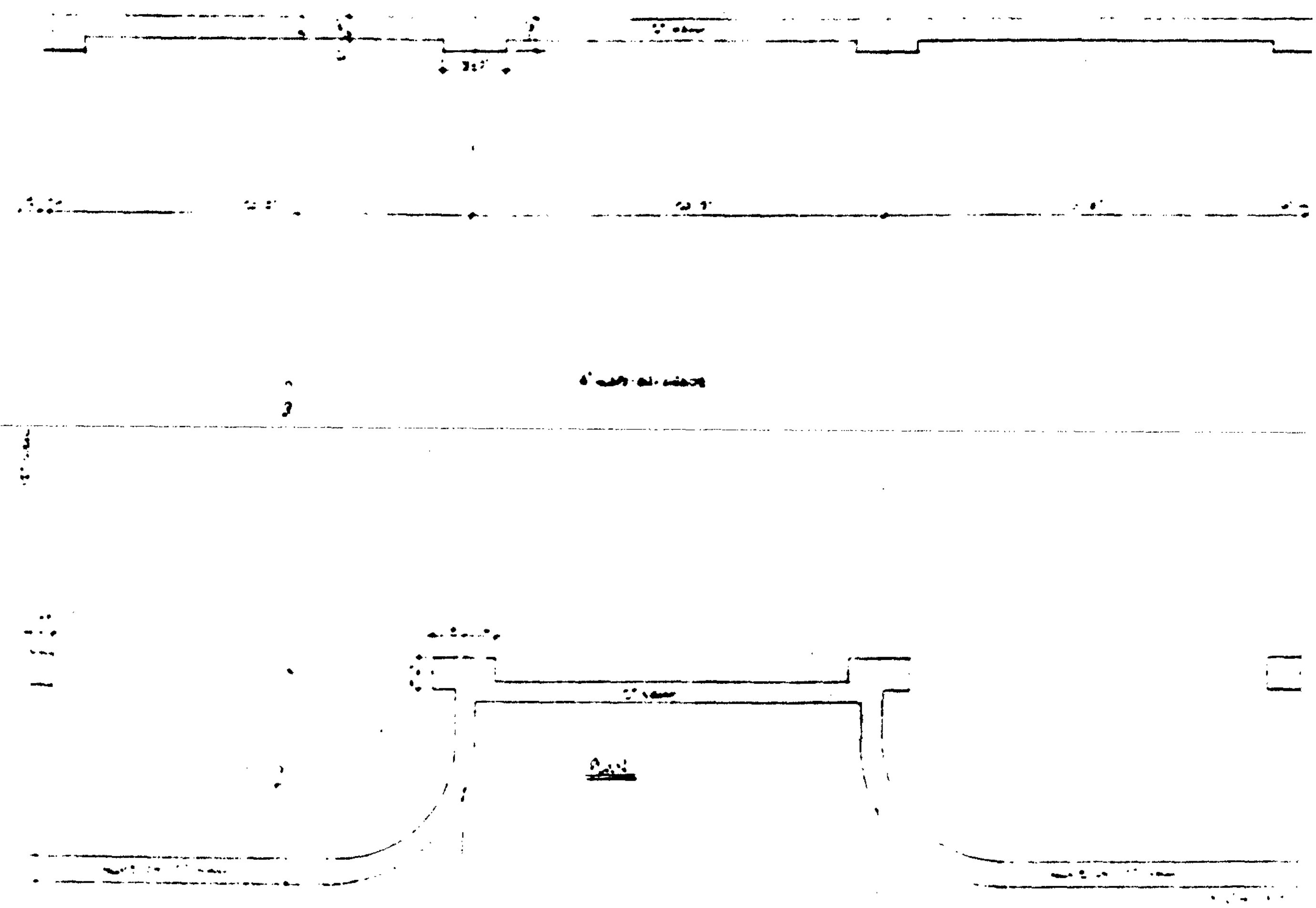


FIGURE 17 -- Structural Details -- Basement Plan

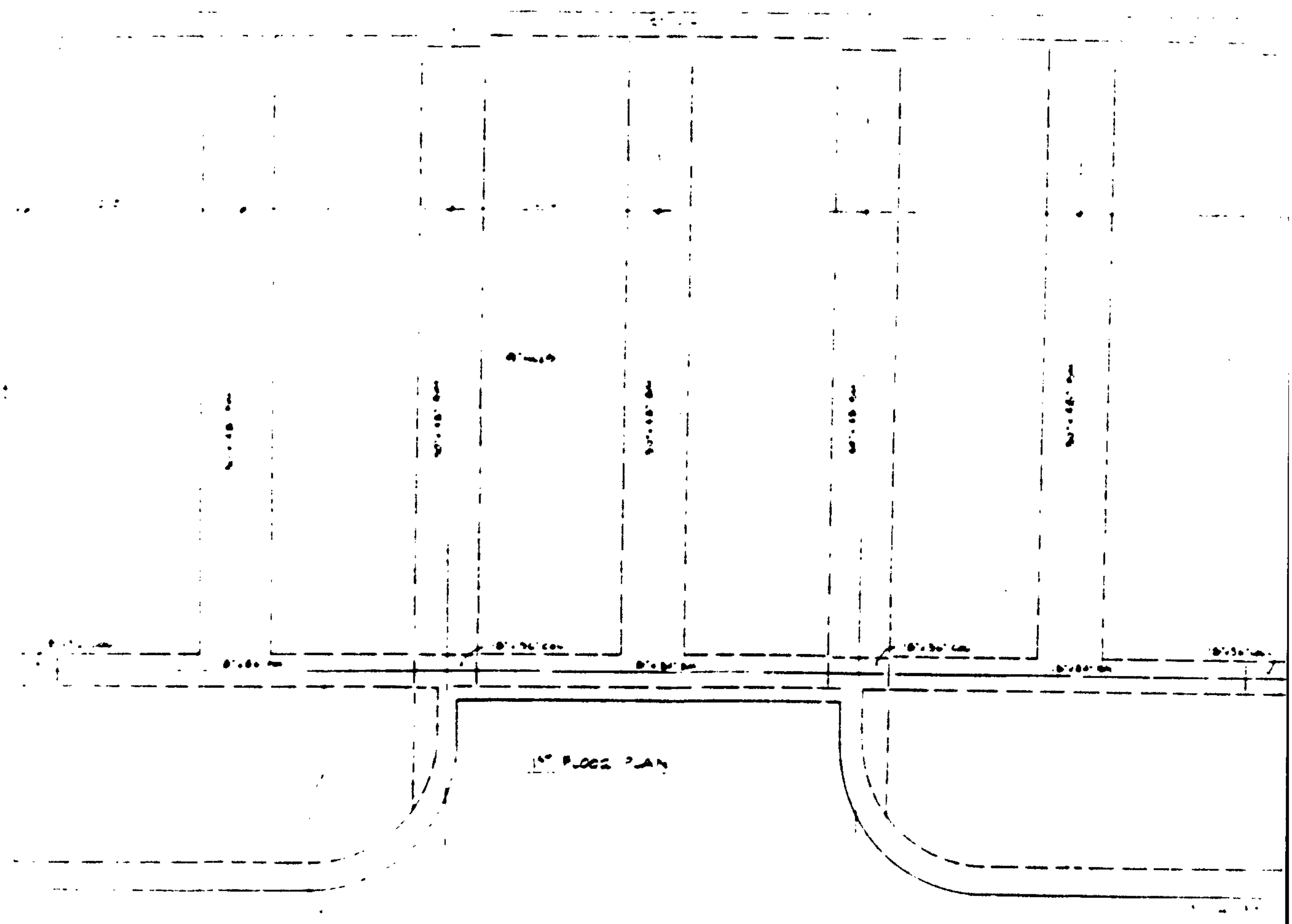


FIGURE 18 -- Structural Details -- First Floor Plan

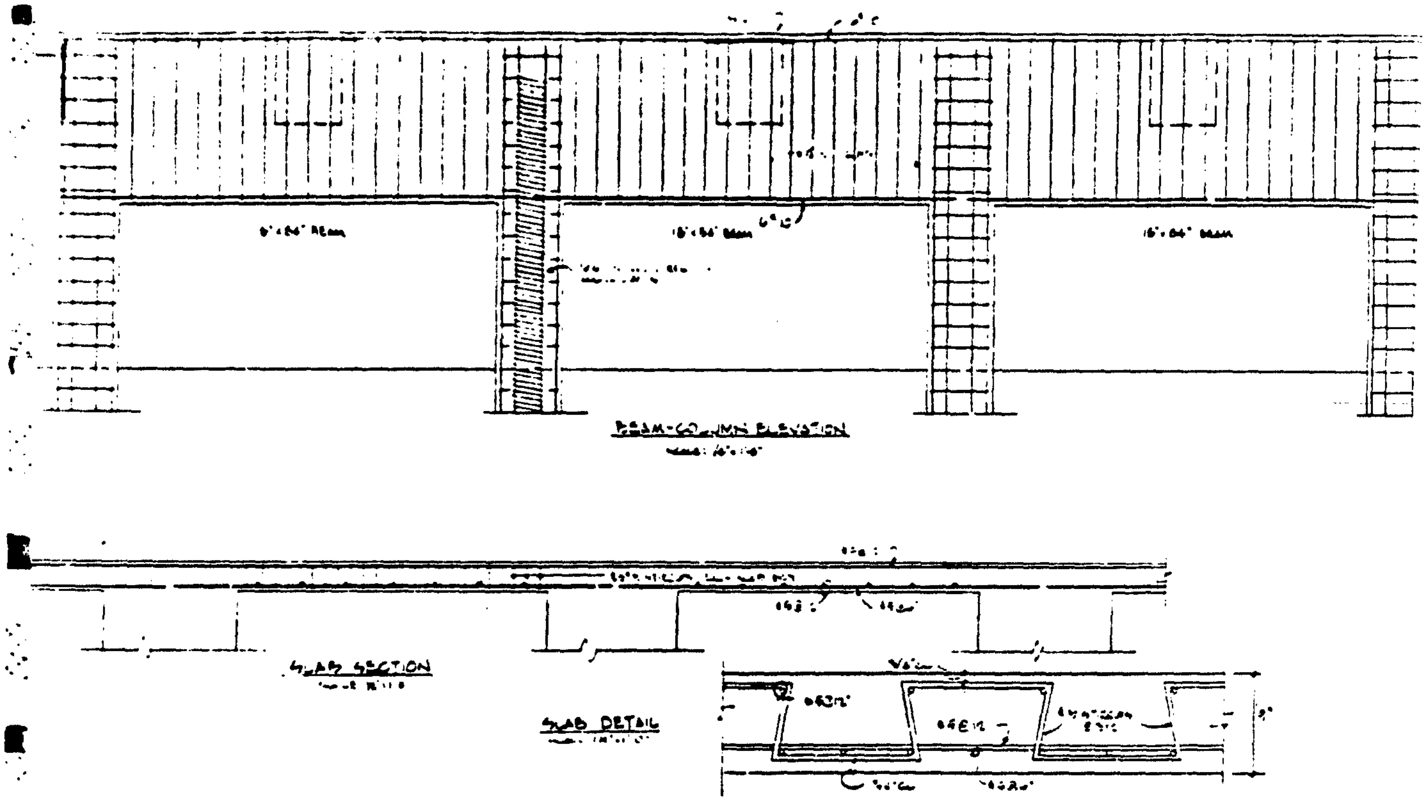


FIGURE 19 -- Structural Details -- Beam Details

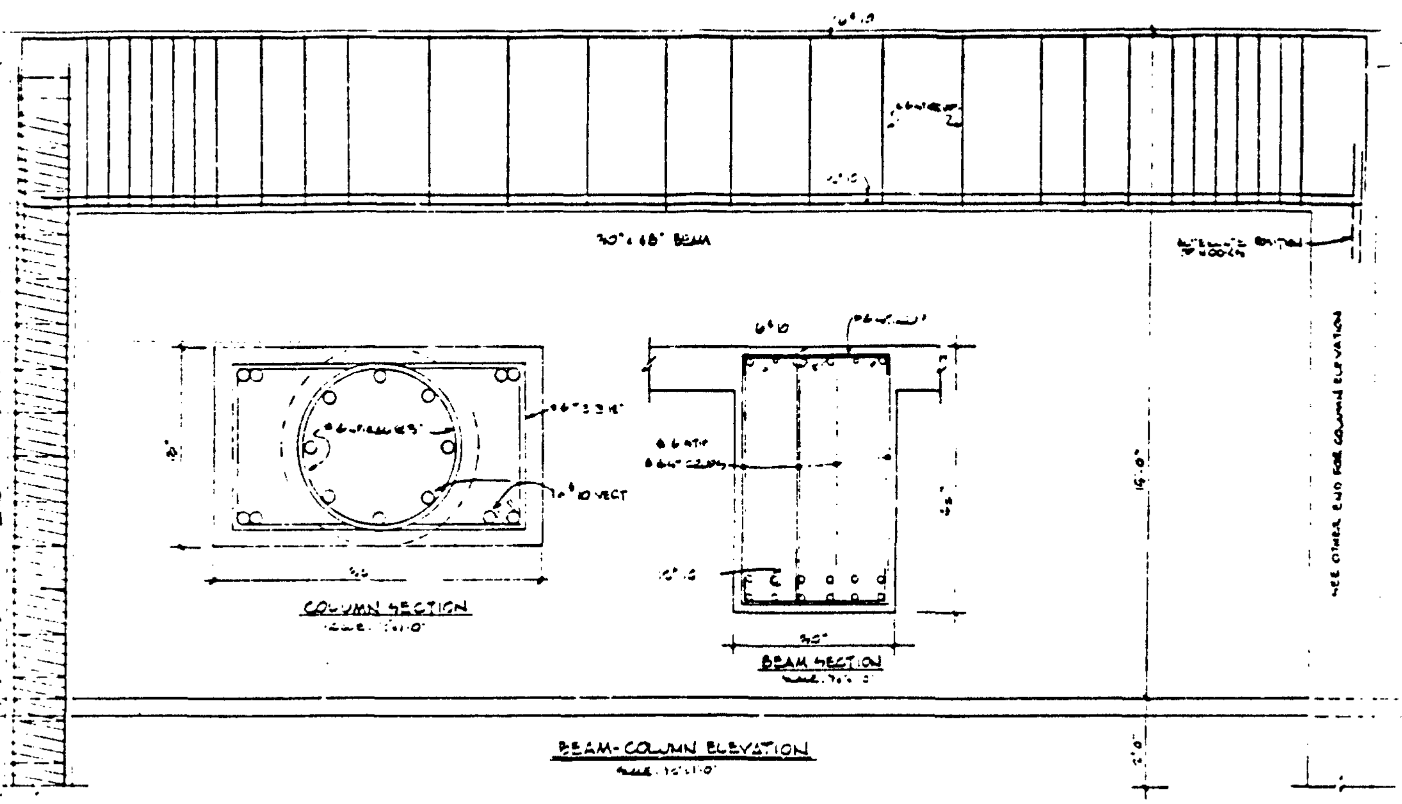


FIGURE 20 -- Structural Details -- Column Details

Turner Construction Company
1331 Pennsylvania Avenue N.W.
Washington D.C. 20004
Telephone 202/393-5100

Turner

August 26, 1983

Mr. Thomas Sachs
Mariani & Associates
1600 20th Street, N. W.
Washington, D. C. 20009

Subject: Blast Resistant Shelter Alternate
National Rehabilitation Hospital

Dear Mr. Sachs:

Please be advised that our price to add the Blast Resistant Shelter is One Hundred Forty Thousand Dollars and No Cents ($140,000.00) and is presented as an Alternate to our National Rehabilitation Hospital GMP Document of June 24, 1983.

The above price is complete and includes General Conditions, Contingency Bonds and Fee.

This price is based on Drawings A-42 and S-13 dated August 19, 1983 prepared by your office. No additional specifications for this work was provided. The drawings are taken to be complete and the price includes no provisions for scope development. We have assumed that this work will be performed consistent with the scheduling requirements of the Project's structural frame. Furthermore, the acceptance of this Alternate will require the addition of one week to the construction schedule.

If you have any questions, please do not hesitate to contact me.

Very truly yours,

TURNER CONSTRUCTION COMPANY

Thomas J. Paci
Chief Estimator

RVS:jw

FIGURE 21 -- Final Construction Bid -- Turner

THE GEORGE HYMAN CONSTRUCTION CO.

4930 DEL RAY AVENUE
BETHESDA MARYLAND 20814

PHONE (301) 986 8100

August 30, 1983

Mr. Theodore F. Mariani, FAIA
President
Mariani & Associates
1600 20th Street, N.W.
Washington, D.C. 20009

Re: National Rehabilitation Hospital
Washington, D.C.

Dear Mr. Mariani:

In accordance with your Mr. Tom Sach's August 19th letter and our conversations with him we offer the following quotations for Alternates #1, #2 and #3 on the above referenced project.

Alternate No. 1 Substitution of Roofing Membrane Material

Alternate No. 2 Addition of Two Elevators

Alternate No. 3 Modify the Structure Surrounding the Auditorium, Audio-Visual/Project Room, and Storage Room to Function as a Blast Resistant Shelter

Add $ 87,000.00

No adjustments will be required to our construction schedule if these alternates are included with the base contract award.

Yours truly,

THE GEORGE HYMAN CONSTRUCTION CO.

A. J. Clark
President

AJC:nm

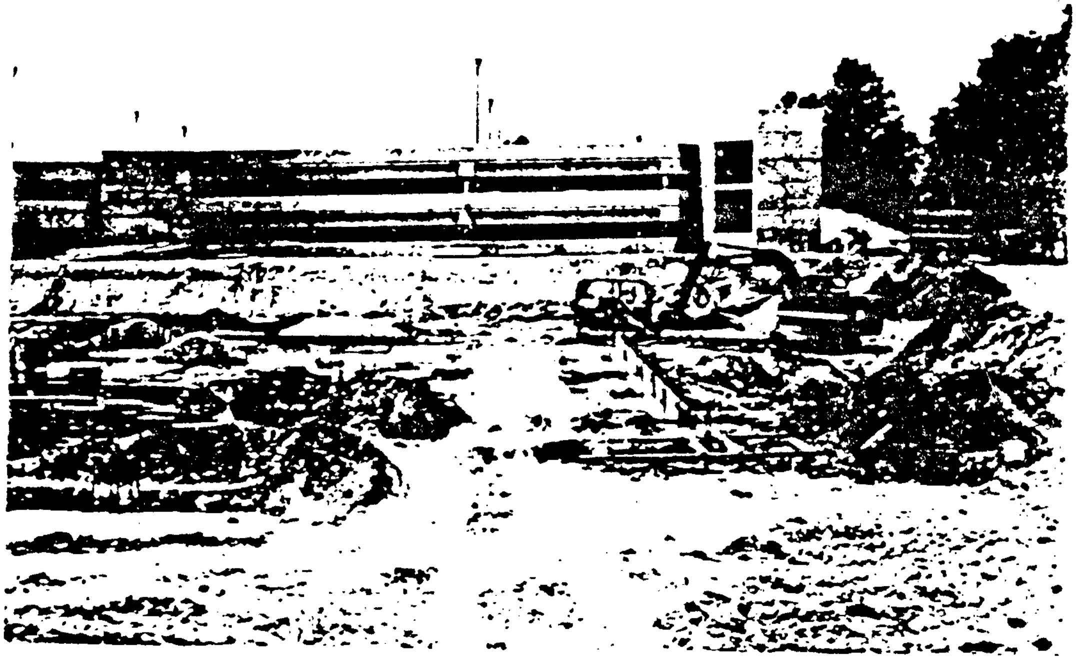
A subsidiary of CEI Construction, Inc

FIGURE 22 -- Final Construction Bid -- Hyman

option was a yes/no decision on construction of the shelter alternate after the National Rehabilitation Hospital had made its decision. The choice turned out to be Turner Construction, the higher shelter bid.

During January of 1984 several other problems occurred. All were unrelated to the blast shelter. There was a change of ownership of the hospital but Turner Construction remained the building construction contractor. There was also a prolonged legal squabble over an adjacent parking structure. Even after the legal disputes were resolved, the construction of the hospital could not be started until a labor strike was settled and construction of the parking garage was completed. The reason for this was that the future hospital site was needed for on-grade parking for the Washington Hospital Center until the parking garage was opened.

Construction. Construction of the hospital was finally begun about mid-May of 1984. AIAF monitored the progress of construction on a weekly basis. A series of photographs (see Figures 23-31) taken on those visits follow.



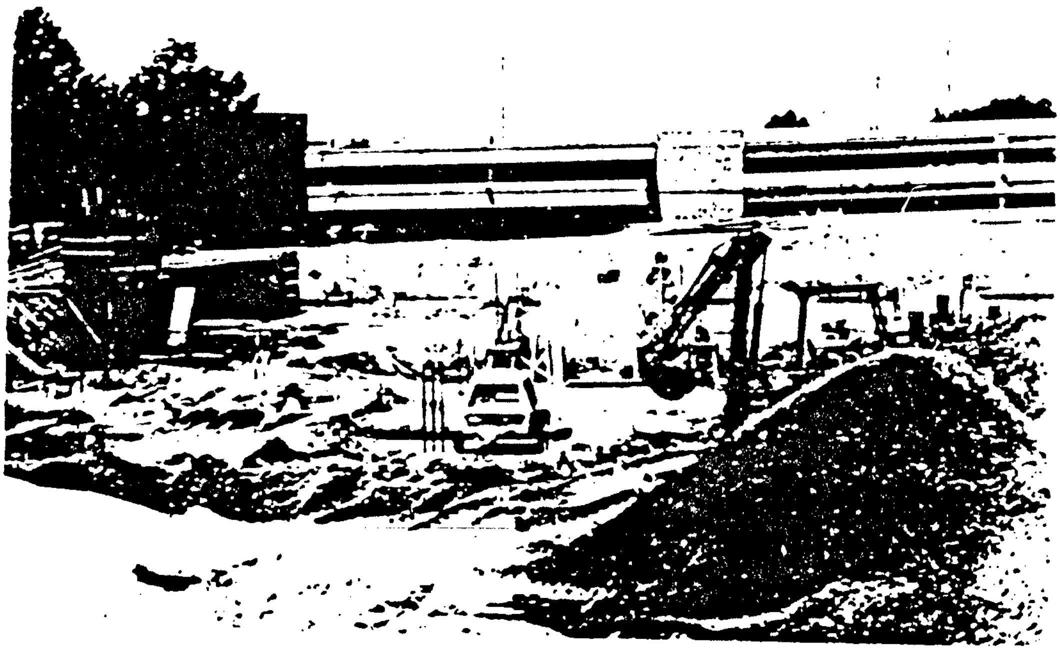
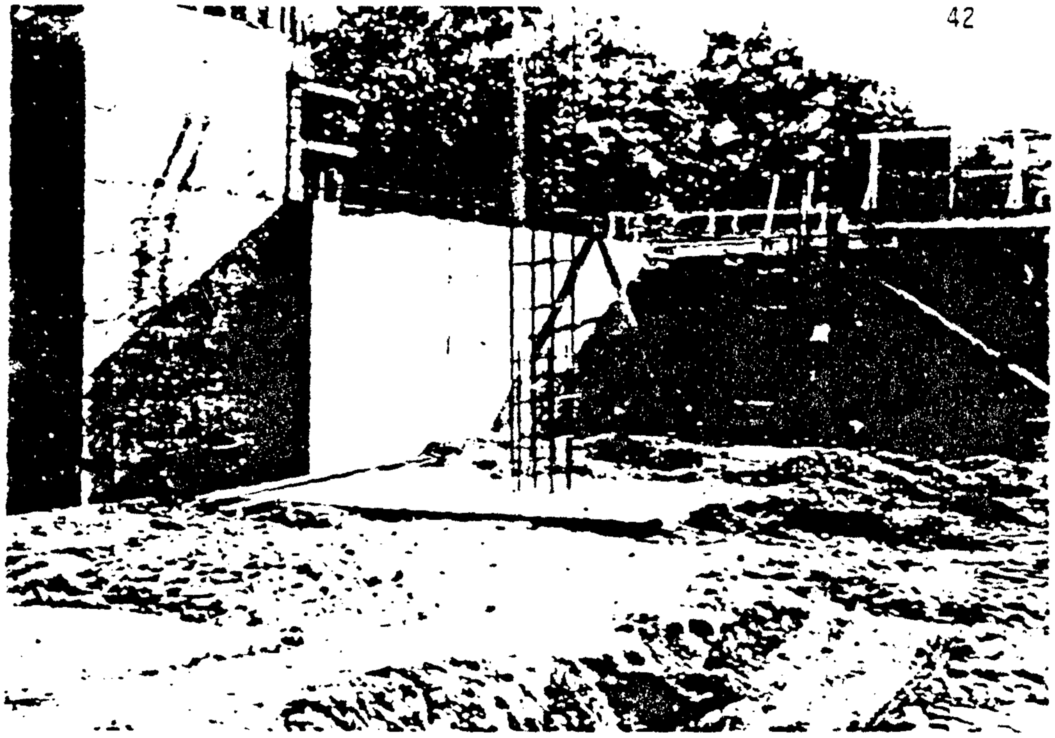
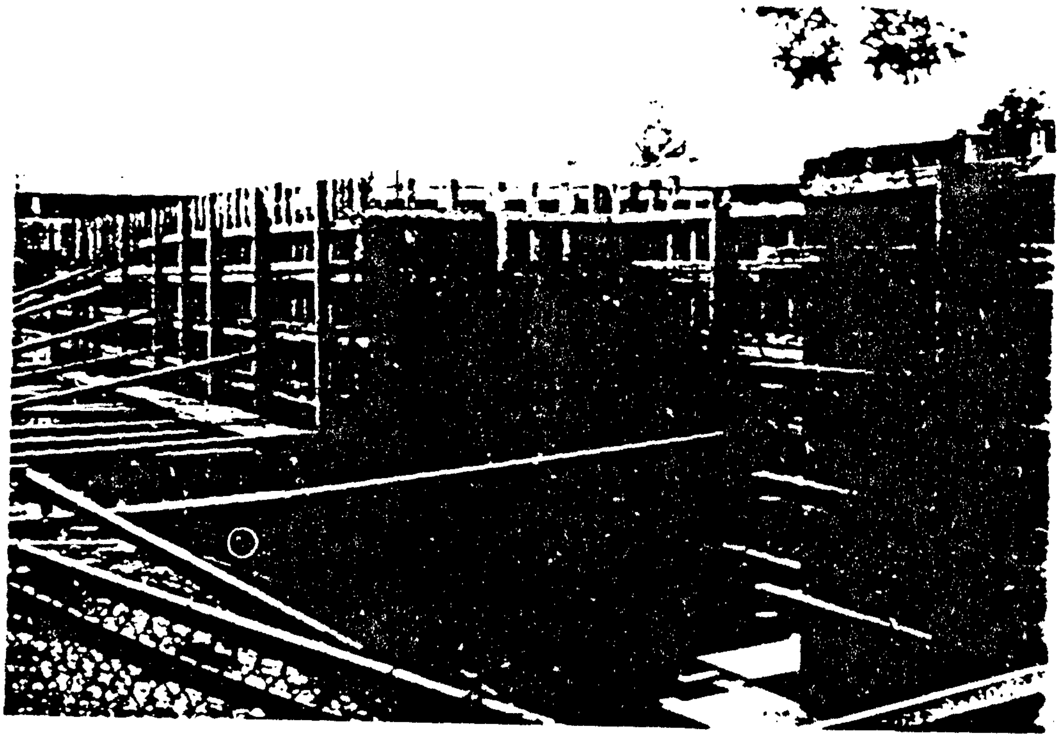


Figure [illegible] -- Hospital Construction -- Site Preparation

FIGURE 24 -- Hospital Construction -- Site Preparation

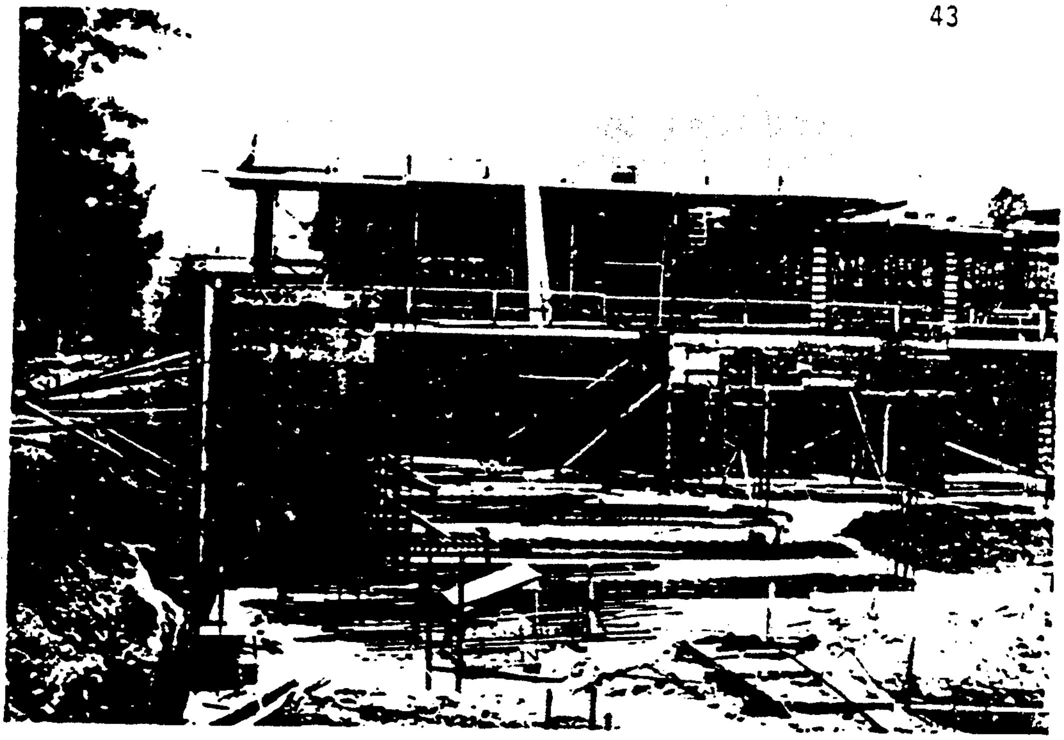


A: Interior



B: Exterior

FIGURE 25 -- Hospital Construction -- South Wall of Shelter



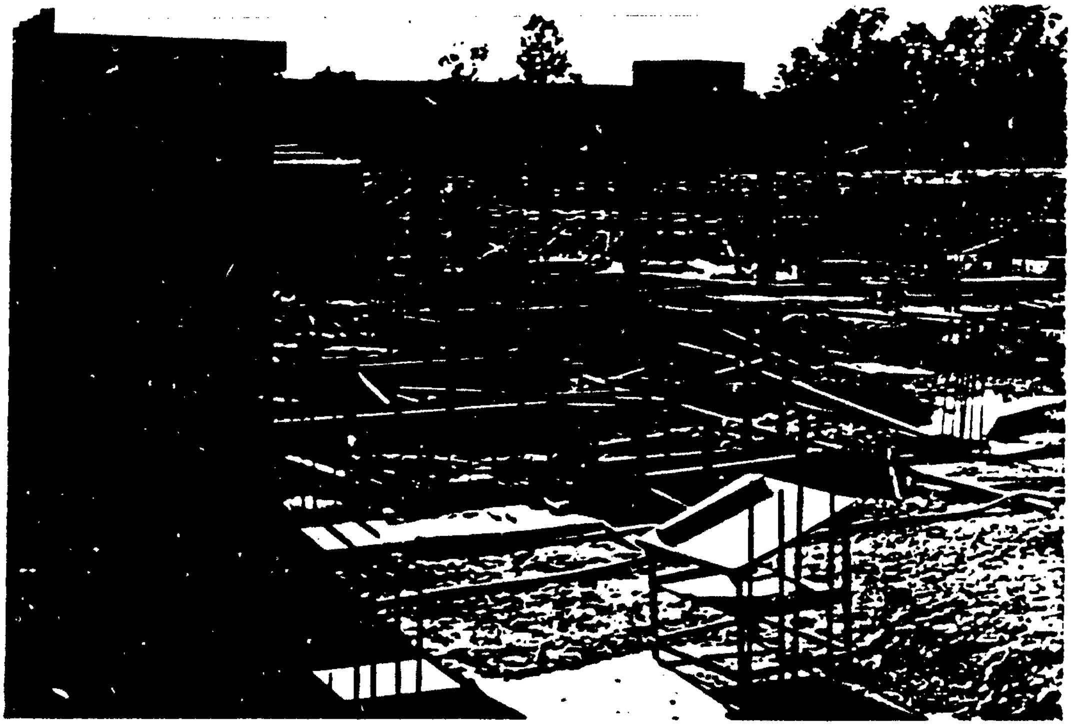


FIGURE 26 -- Hospital Construction -- Footing Preparation

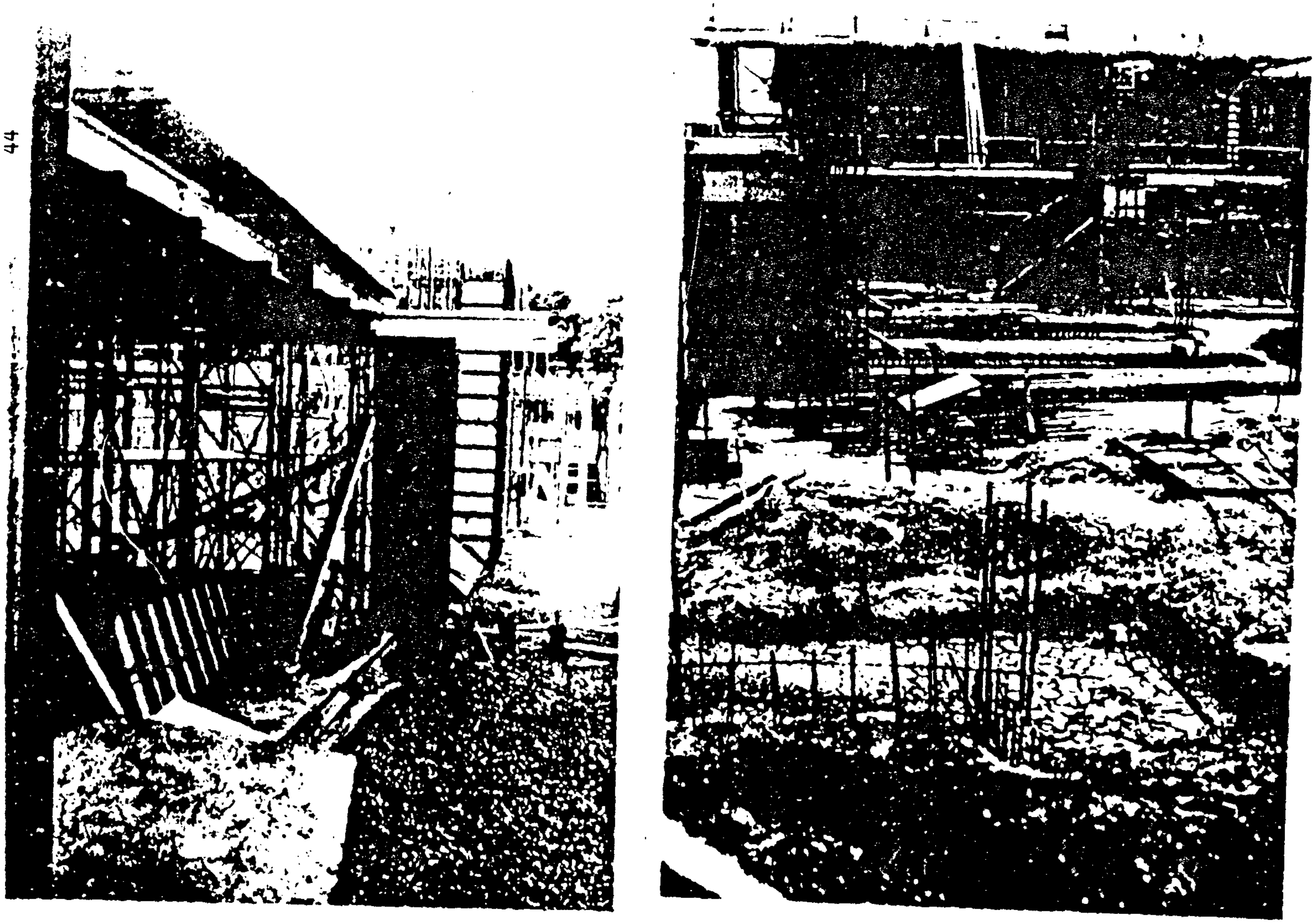


FIGURE 27 -- Hospital Construction -- Adjacent Story

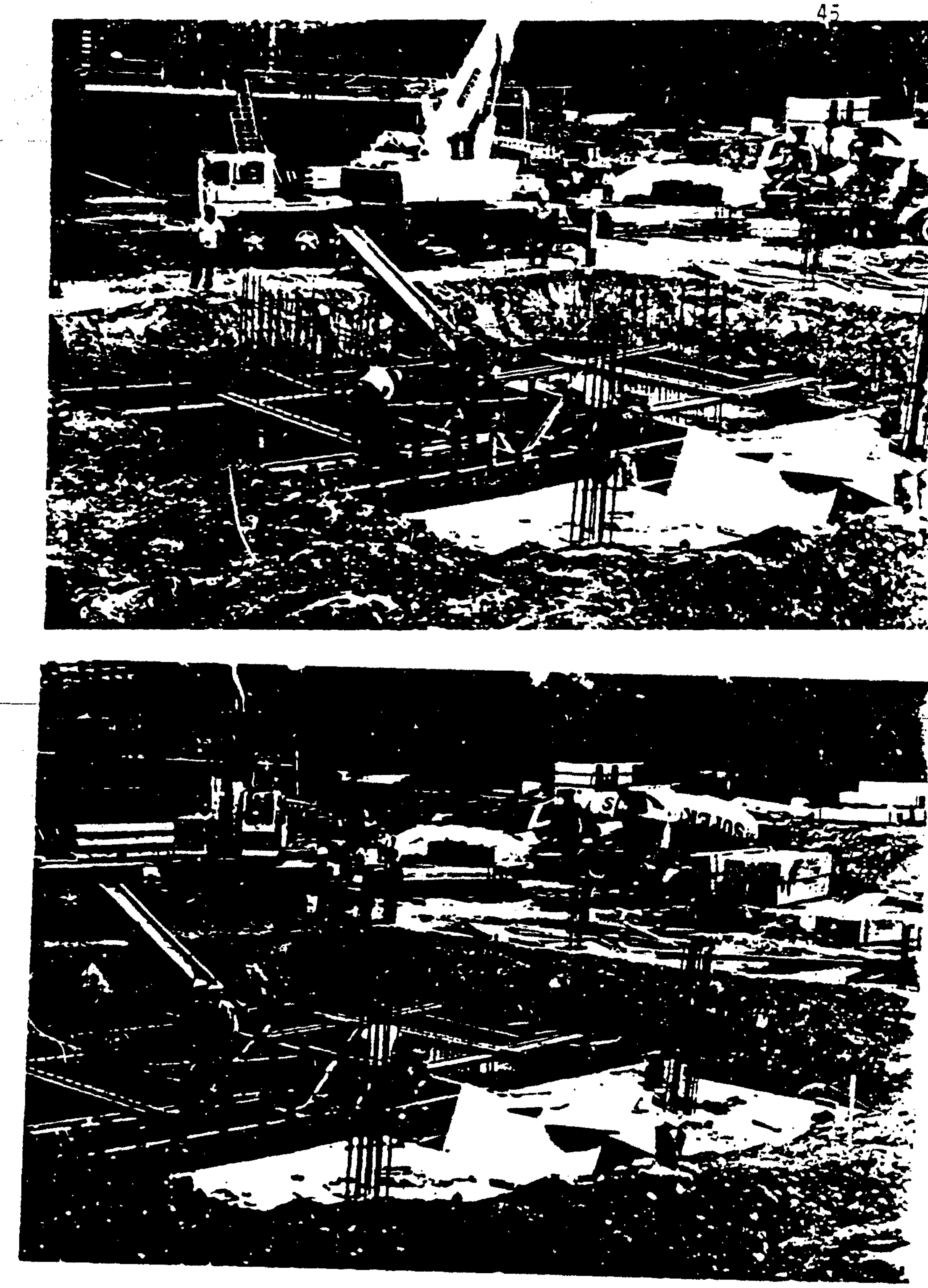


FIGURE 28 -- Hospital Construction -- Concrete Formwork

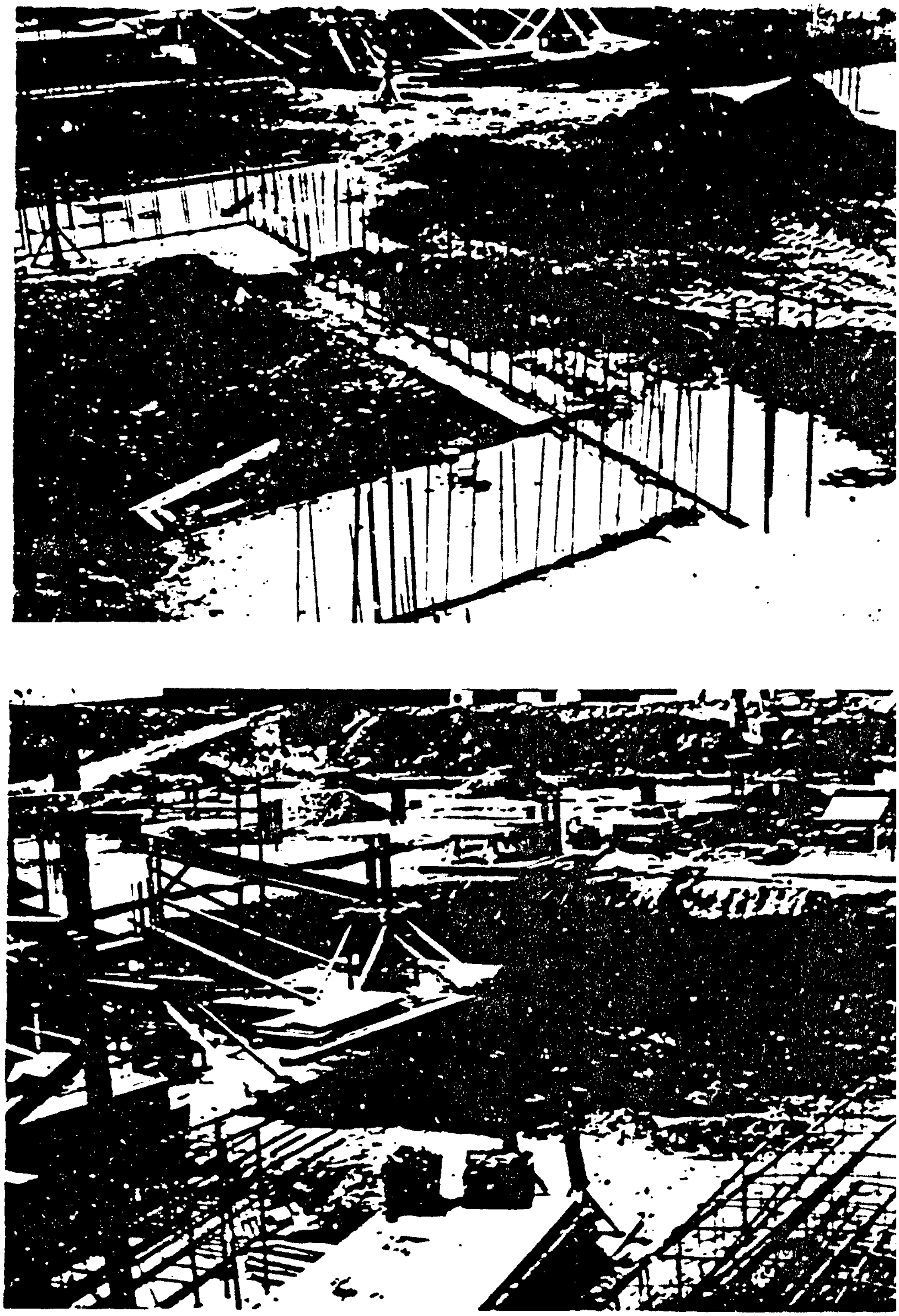


FIGURE 29 -- Hospital Construction -- Footing Poured

FIGURE 30 -- Hospital Construction -- Column Reinforcing Steel

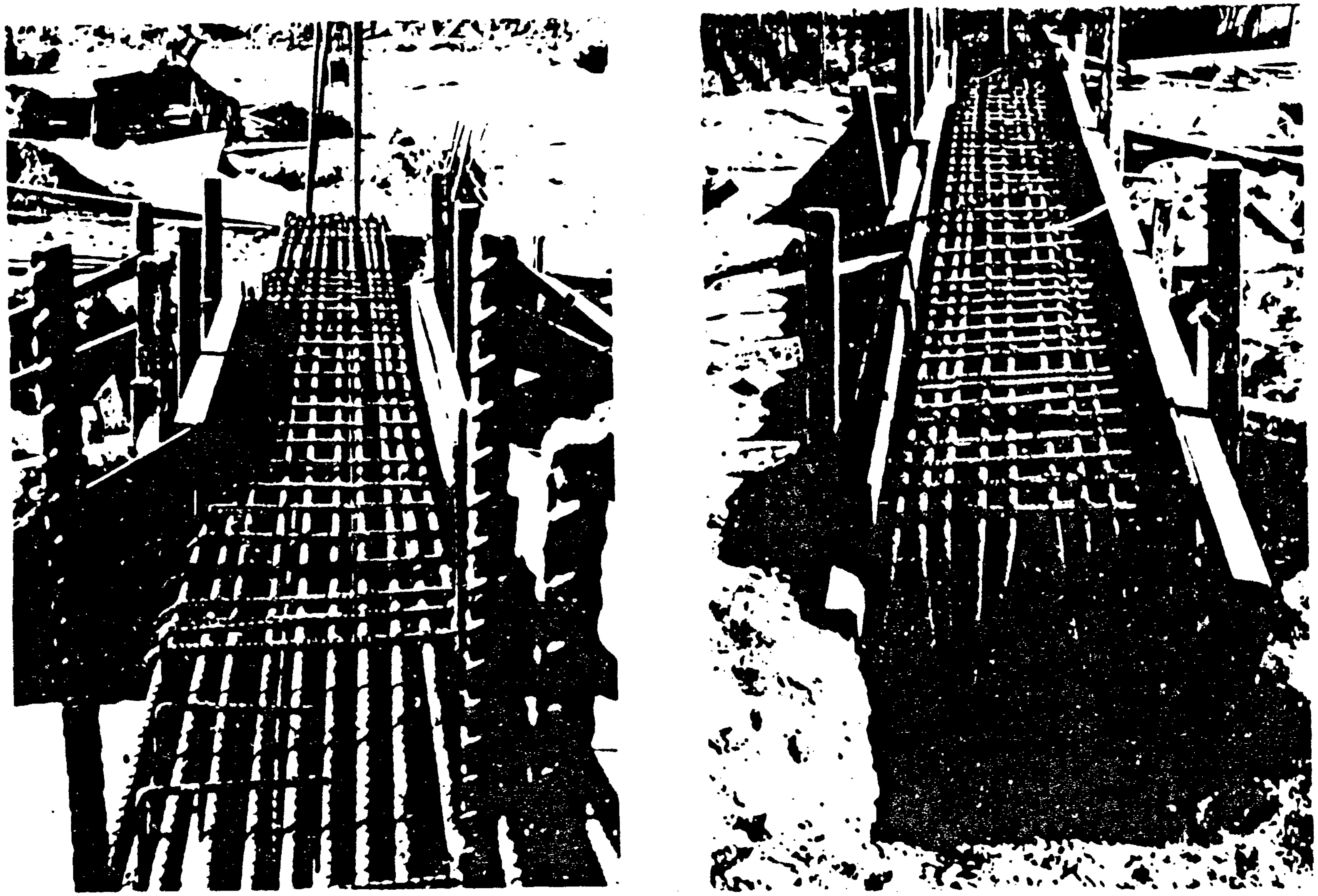


FIGURE 31 -- Hospital Construction -- Beam Reinforcing Steel

A two-installment construction contract for $140,000 was written with the National Rehabilitation Hospital, Inc. The first payment for 95 percent was payable at the completion of all concrete work, and the remaining 5 percent at completion of all work on the blast shelter.

Two construction related problems occurred but both were easily resolved. The first problem involved the fact that the shop drawings for the reinforcing steel were completed after the major steel order had been placed. Rather than risk a possible construction delay awaiting the specialized reinforcing steel, the decision was made to redesign the shop drawings to use commonly available steel. A second problem occurred when a concrete footing was poured from non-shelter construction documents which left the footing one foot higher than it should have been. The resolution was a redesign of the detail to accept a beam above the new footing (see Figure 32). No other significant problems were encountered during the construction. There was difficulty with ground water in an area proximal to the shelter footings, but this was not attributable to the shelter design.

AIAF performed a walk thru inspection of the shelter on September 25, 1984, with representatives of Mariani and Associates and Turner Construction. Construction of the blast shelter was virtually complete at that time. Major observations on the construction process were the following:

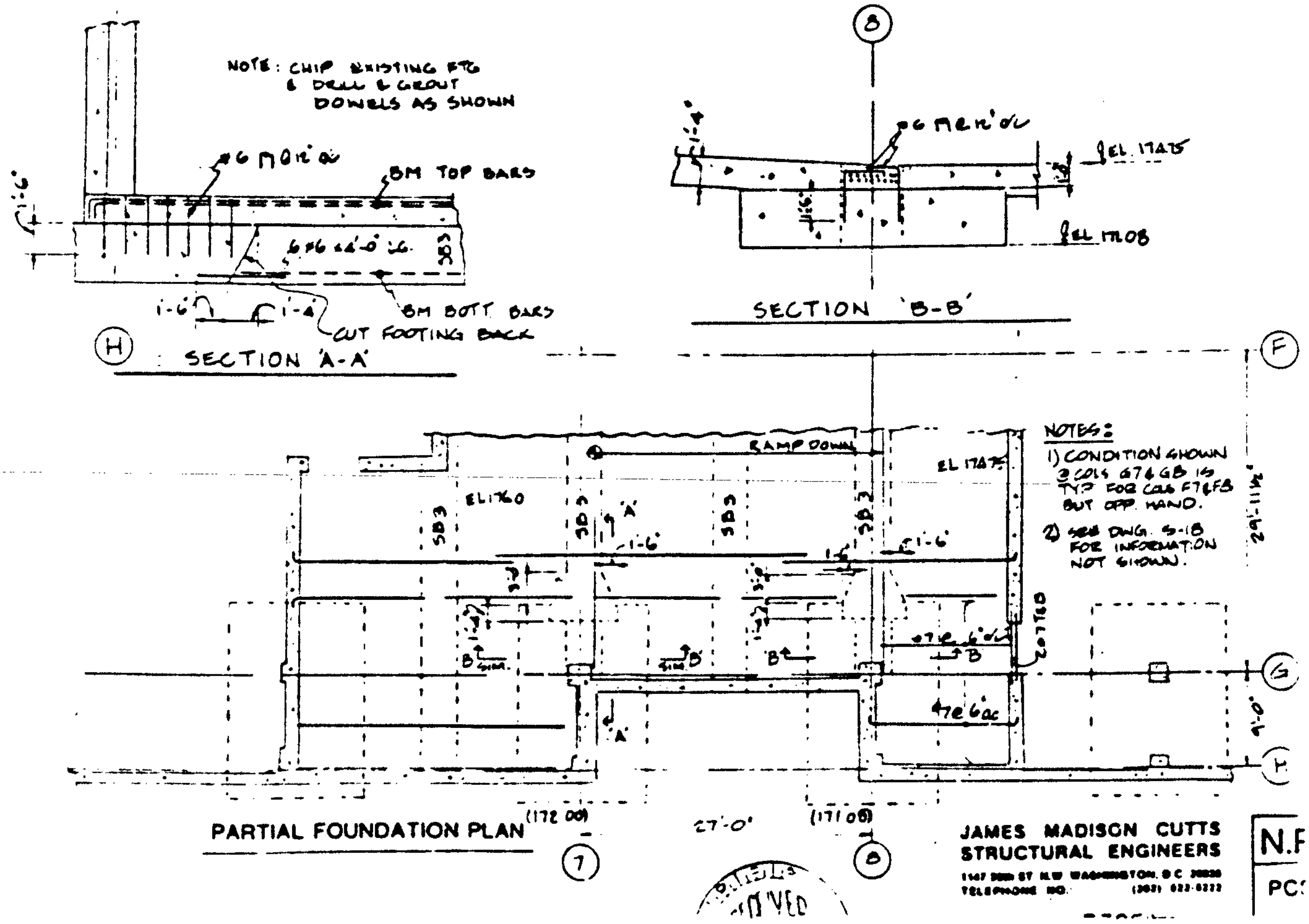


FIGURE 32 -- Beam/Footing Connection Details

o Construction difficulties exist with wall penetrations and beam corners which do not fit flush with other building elements.

o The scheduling of concrete pours requires careful planning and special attention.

o Careful planning and additional time is required for building up reinforcing steel during shelter construction.

o Blast shelter construction did not constitute an extraordinary problem. It was rather one more layer of complexity to be coordinated into the overall process.

Use of TR-20 - Volume 4 and Other Design Guidance:

One of the objectives of the project was to evaluate the usefulness of existing slanting design guidance materials. A national program of blast shelter construction which would be necessary to sustain the critical workforce concept would require design guidance materials which were understandable and immediately usable by architects and engineers without prior shelter design experience. Architects, most of whom would not have shelter design expertise, would certainly be involved in the process of incorporating blast shelters into otherwise typical buildings in a large-scale construction program.

In recent years the practice of architecture has increasingly become a problem of leading a team of experts. The architect's task is to communicate effectively with each specialist and ensure that each separate agenda is incorporated into the overall program for the building and addressed adequately as the design proceeds. The ideal design manual would provide the architect with a conceptual overview of what slanting design is all about. Such a document should contain visual presentation of architectural concepts. It is difficult to overstate the importance of visual, graphic thinking for architects.

The findings of the architects and engineers working on this project are that Volume 4, TR 20, "Protective Construction", and the other documents used are inadequate and provide insufficient guidance. The personnel at Cutts Engineers had no prior shelter design experience and became quite frustrated, even angry, at some of the difficulties they experienced. Don Neubauer, on the other hand, did have prior shelter experience. On the positive side, Mr. Neubauer commented that Vol.4 TR 20 is superior to other design guidance materials (military documents) he has used, especially when designing something out of the ordinary. Comments from both structural engineers follow at the end of this section (see Figures 33-34).

The existing reference documents (see Figure 35) were judged to be more textbooks than design manuals and had other problems as well. Definitions and terminology were inconsistent. Several references were needed to clarify symbology. The layout of Vol.4, TR 20 is quite cumbersome, and requires considerable searching and flipping back and forth. All of these problems increase time of use and consequently design costs.

The ideal design manual would be a single, self-contained reference source. It should present straightforward examples, problems and many charts and tables to assist the busy, inexperienced designer.

NEUBAUER

November 12, 1982

Mariani and Associates
1600 20th St., N.W.
Washington, D.C. 20009

Re: Blast-resistant design
National Rehabilitation Hospital

Gentlemen:

Following are comments that we offer regarding the use of "Protective Construction" TR-20 (Vol 4) May 1977 in the design of blast resistant structures. There is no particular significance in the order of the comments and some come as a reflection about the use of the manual. the manual is competent in presenting a very complex problem in one volume. The comments are not intended to give a negative value to the manual.

1. The examples given are rather irrelevant. It is assumed that the manual will be used to harden certain areas of a building and it would be most helpful to have examples that relate to the same. Also it is doubtful that an attempt would be made to harden a structure above ground to act as a blast resistant structure, so the number of relevant sample calculations need not be large.

2. An index would be very helpful. Considerable time was spent in searching both backwards and forwards in the manual to refresh ones understanding of the meaning of a certain letter. The meaning of symbols was not always given when the symbol was first used. This is a deficiency in the majority of technical books. Some symbols are used in the text and not noted in the list of symbols.

3. Some symbols were casually explained further on in the manual which makes use of the manual time consuming. Some had two different meanings and which one was being used was not always that apparent.

4 Sometimes the lower case of symbols was used in the text and upper case in the charts or graphs. In some instances elsewhere in the text the upper and lower case have symbols that have different meanings. This creates an uncertainty in that one is really never certain that the correct definition is being used. If one designs a lot of hardened structures on a frequent

basis, these uncertainties resolve themselves. But it is suspected that the vast majority of architects and engineers will do so on a very infrequent basis and each subsequent problem would result in beginning again with the studying of the manual. Design costs for hardened structures will be very high.

5 Fortunately copies of the principal references were in hand which were of help. It is doubtful that more than a very small percentage of architects and engineers would be so equipped. Therefore the manual needs to be more explicit even to the arithmetic so that the time spent using the manual. The references were used to determine more clearly the meaning of some of the symbols.

6 Very few architects and engineers these days have the time to devote to design projects that is really needed. The lack of time usually results from insufficient fees and insufficient expertise, so those that use the manual will have to follow it "blindly" without fully understanding the design process. Under these conditions, in order to minimize errors, all symbols, formulas and procedures have to be readily apparent and not involve referring to other steps behind or ahead in the text unless those steps are specifically annotated as to page and paragraph.

7. It was noted in the design problem for the National Rehabilitation Hospital that there were many design details that were not considered in the manual. In doing blast resistant design, the problem should be formulated to conform with various charts and tables rather than trying to interpolate the charts and tables to a problem that is not "simple". Such a suggestion should be noted in the introduction. Structurally dynamic design is much more an art rather than a science. But for most engineers and architects, the more science and the less art, the better the solutions will be with respect to costs and performance. This means having problems that are comfortably within the range of the manual's tables and charts.

8 The best way to obtain comments on the manual would be to sit down with a user and review the manual page by page. Hopefully the authors will not be defensive for if they are not as much value will be derived from the exercise as would be otherwise.

Very truly yours,

Donald J. Neubauer

Donald J. Neubauer

copy file
DJN: iqn

FIGURE 33 -- TR20 (Volume 4) Evaluation -- Neubauer

James
Madison
Cutts

[illegible]
20036
822 8[illegible]

March [illegible], 1983

REPORT ON USEABILITY
OF
PROTECTIVE CONSTRUCTION MANUAL
(TR-20 Vol. 4)

In an attempt to use the Protective Construction Manual (TR-20) as a primary and only source to design a blast resistant shelter, it was found that the manual by itself is inadequate.

The following are major points in design which, in our point of view, were lacking in the manual and require more attention and elaboration:

1. Loading Terms:

The loading definitions were found to be very confusing. The manual should elaborate more on definitions of loading and loading on the structure to provide a simplified means of understanding of loading for the designer.

2. Safety Factor:

The application of load factor in respect to ultimate strength design is not clearly explained. It should be noted that load factor in respect to ultimate strength design equal to 1.0.

3. Modes of failure:

Since the design is controlled by three modes of failure (shear, diagonal tension and flexure), the manual should elaborate on approach to design through the three modes of failure. The manual should familiarize the designer with behavior of the structure in the three different modes. Also, the importance of choosing the appropriate ductility ratio for each mode of failure.

4. Ductility:

Since ductility is of great importance in blast resistant design, the manual should elaborate more on the subject of ductility. It should familiarize the designer with the importance of assuming the required ratio of ductility for each mode of failure and explain why. Also, it should be noted that the formula $\mu = \frac{1}{(P-1)}$ is only a definition and it is [illegible] an equation.

5. Dynamic Analysis:

The manual should provide complete practical examples for dynamic analysis and explain what results to look for in dynamic analysis. Since this portion of design is the most complicated and time consuming, the manual should concentrate on use of charts and tables through sample problems.

6. Slab on grade:

Since design of slab on grade requires careful attention, the manual should explain the procedure and assumptions which should be used in design of slab on grade (floating slab or fixed).

7. Rebound resistance:

The calculation of rebound steel requires more elaboration.

8. Preliminary & final design:

In final design calculation, it should be noted that convergence of only ductility ratios for flexure mode is required and ductility ratios used in shear and diagonal tension are to provide only a means of checking resistance for those modes of failure.

9. Typical details:

Typical details & spacing for web reinforcing should provide guidance for size and spacing of ties beyond the distance of 4d from support. It should also be noted that since the slab is allowed to have large deflections, the presence of web reinforcing also provides a means of securing top and bottom reinforcing in place during deflection and rebound.

10. Charts:

Since charts and design aids in blast resistant design can save the designer a considerable amount of time, the manual should include all the charts and design aids available for blast resistant design.

General:

In general the manual is an informative text book in the field of blast resistant design. However, it is inadequate and impractical to be used as a design manual by an average structural engineer, especially for one with no back ground in dynamic analysis and design.

FIGURE 31. TR-20 (Volume 4) Evaluation -- Cutts

REFERENCE DOCUMENTS

TR-20 (VOLUME 4) PROTECTIVE CONSTRUCTION. Defense Civil Preparedness Agency. May 1977.

DESIGN OF STRUCTURES TO RESIST NUCLEAR WEAPONS EFFECTS. The Committee on Structural Dynamics of the Engineering Mechanics Division. Engineering Practice Manual No.42, 1962.

THE EFFECTS OF NUCLEAR WEAPONS. S. Glasstone & P. Dolan (eds.). Prepared by U.S. Department of Defense and U.S. Department of Energy. Washington, DC: U.S. Government Printing Office, 1977.

MAXIMIZING PROTECTION IN NEW EOS'S FROM NUCLEAR BLAST AND RELATED EFFECTS: GUIDANCE PROVIDED BY LECTURE & CONSULTATION. NTIS Accession Number AD/A 039 499, September 1976.

PRINCIPLES AND PRACTICES FOR DESIGN OF HARDENED STRUCTURES. N.M. Newmark & J.D. Haltiwanger, Department of Civil Engineering. Technical Document Report Number AFSWC-TDR-62-138, December 1962.

REINFORCED CONCRETE DESIGN. Wang & Salmon. New York: International Textbook Company, 1973.

SLANTING IN NEW BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: A CONSOLIDATED PRINTING OF FOUR TECHNICAL REPORTS. NTIS Accession Number AD/A 023 237, October 1975.

FIGURE 35 -- Existing Reference Documents

Model Test:

Two models (at one-fifth scale) of the blast shelter at the National Rehabilitation Hospital were constructed. These models were built by Waterways Experiment Station under a separate contract to the Federal Emergency Management Agency. These two models were exposed to a simulated nuclear explosion at White Sands, New Mexico, in July of 1985. The following photographs were taken in New Mexico at the blast test (see Figures 36-45). One model was placed at a location so that it was exposed to the 15 psi overpressure for which the actual shelter was designed. The other model was placed at a location so that it was exposed to 50 psi overpressure. No above-grade structure was included on either model which further increased the vulnerability to structural damage, because this was taken into account in the actual design calculations.

The model exposed to 15 psi overpressure suffered no apparent structural damage (see Figure 43). The model exposed to 50 psi overpressure suffered only minor damage in the form of hairline cracks. The cracks may be seen on the photographs (see Figures 44-45). The cracks were traced with a marker to highlight their location, but the result is that the damage appears more serious than actual. The apparent conclusion is that the design guidance provided by "Protective Construction", TR 20, (Vol.4) leads to partial overdesign for shelter structures intended to withstand 15 psi overpressure.

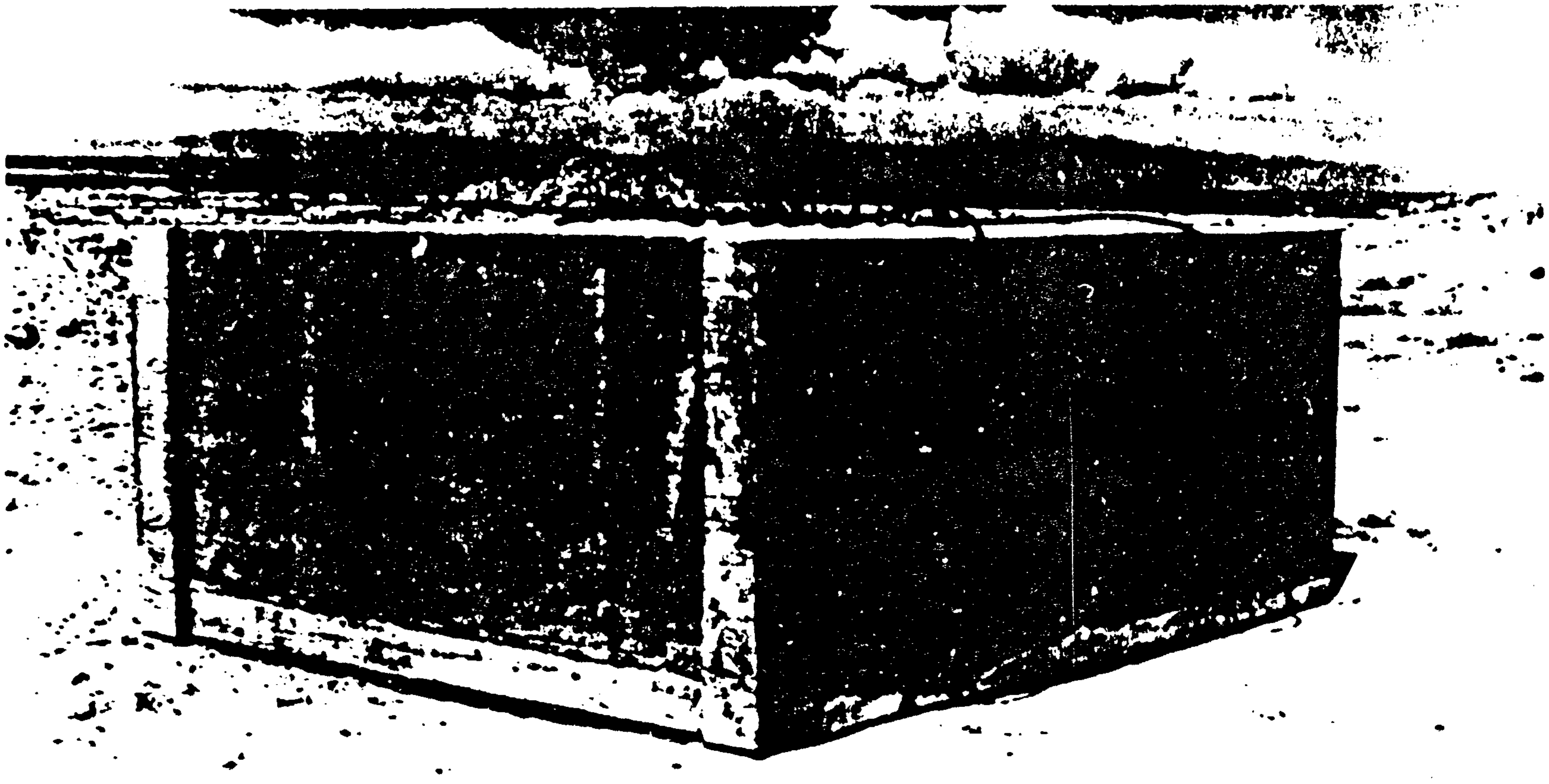


FIGURE 36 -- Blast Test of Model -- Front & Side of Shelter

FIGURE 37 -- Blast Test of Model -- Rear & Side of Shelter

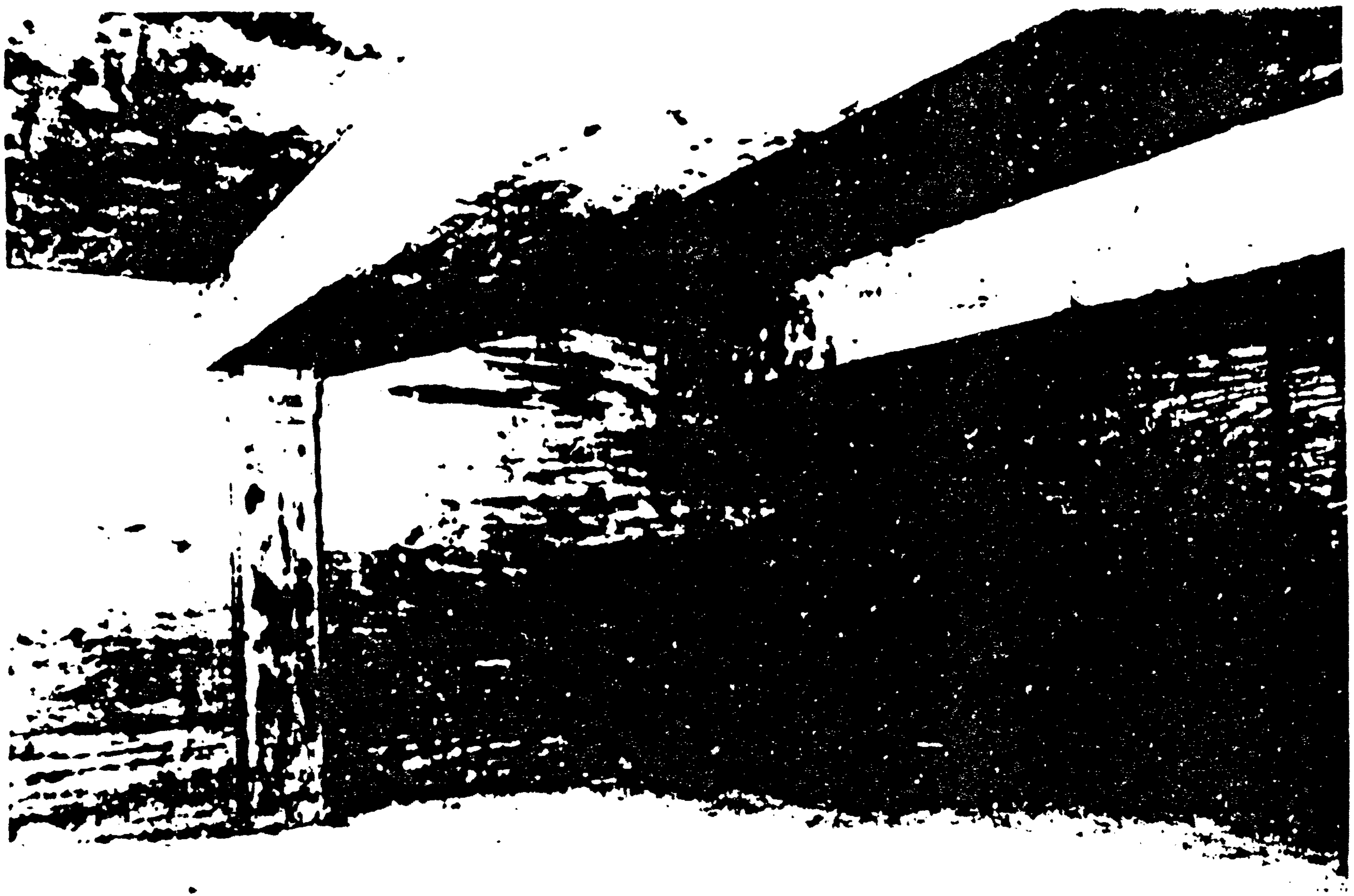


FIGURE 33 -- Interior of Hospital Shelter

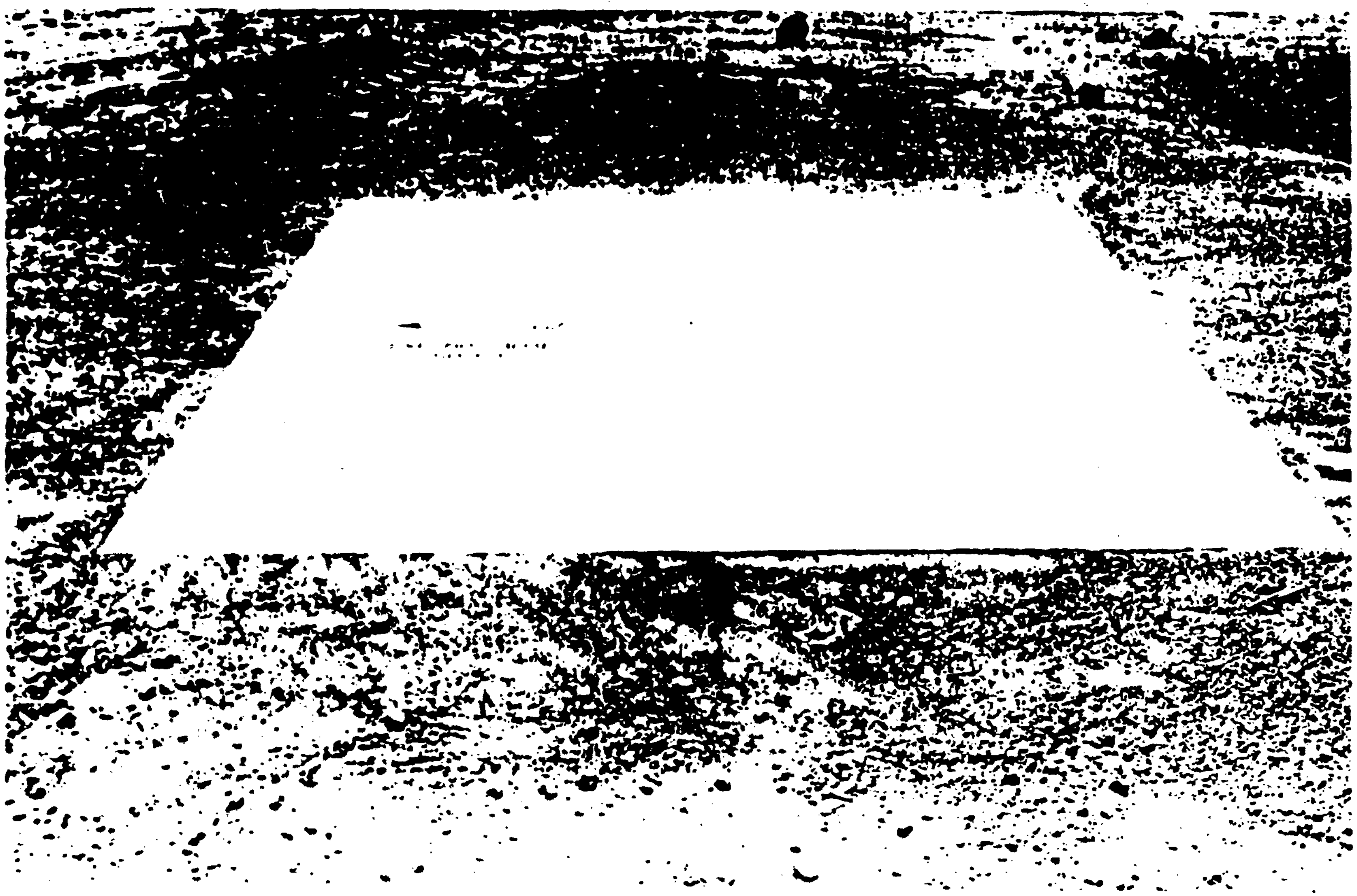


FIGURE 41 -- Hospital Shelter at 50 psi Level -- After Back Filling

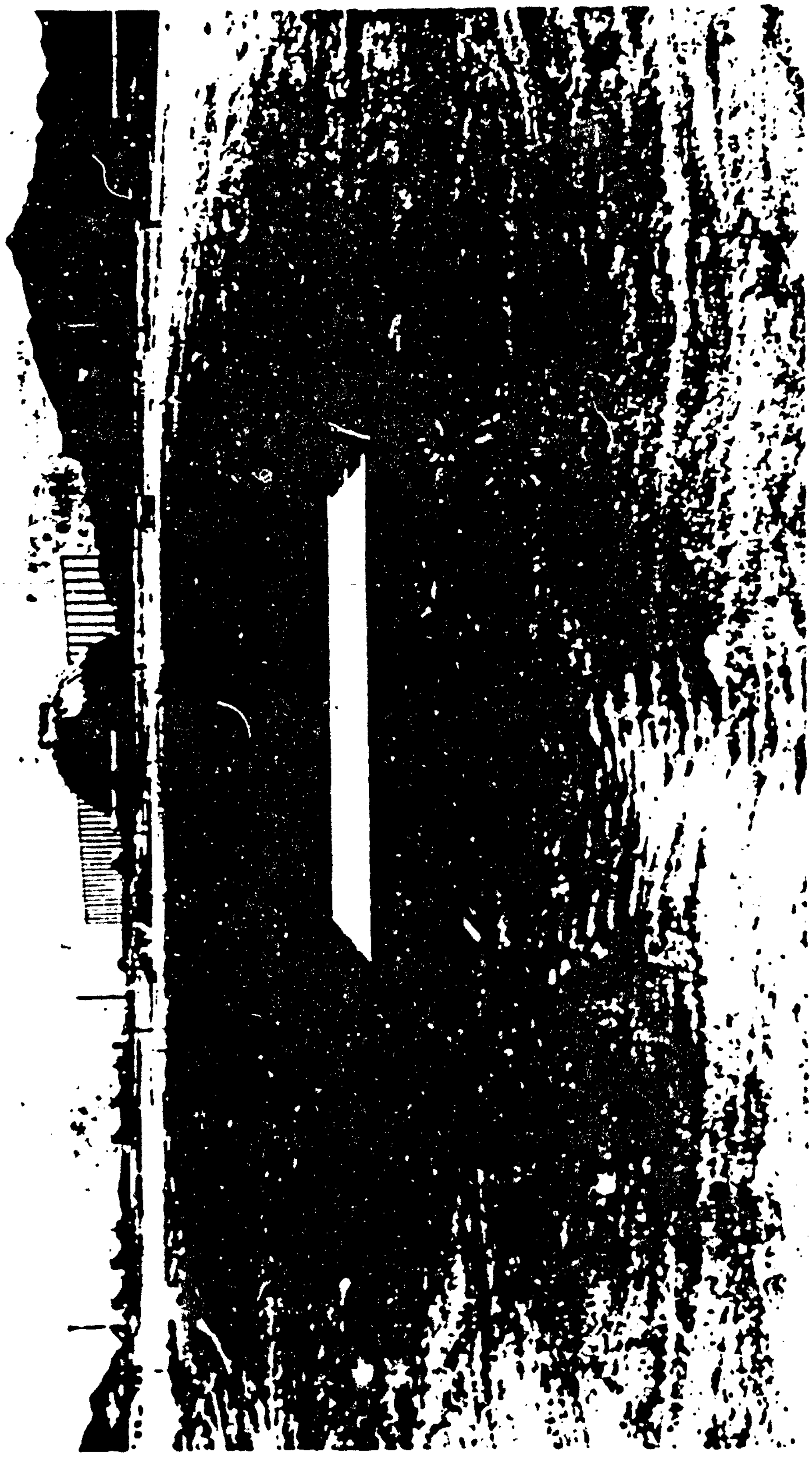


FIGURE 42 -- View of Hospital Shelter at 50 psi Level

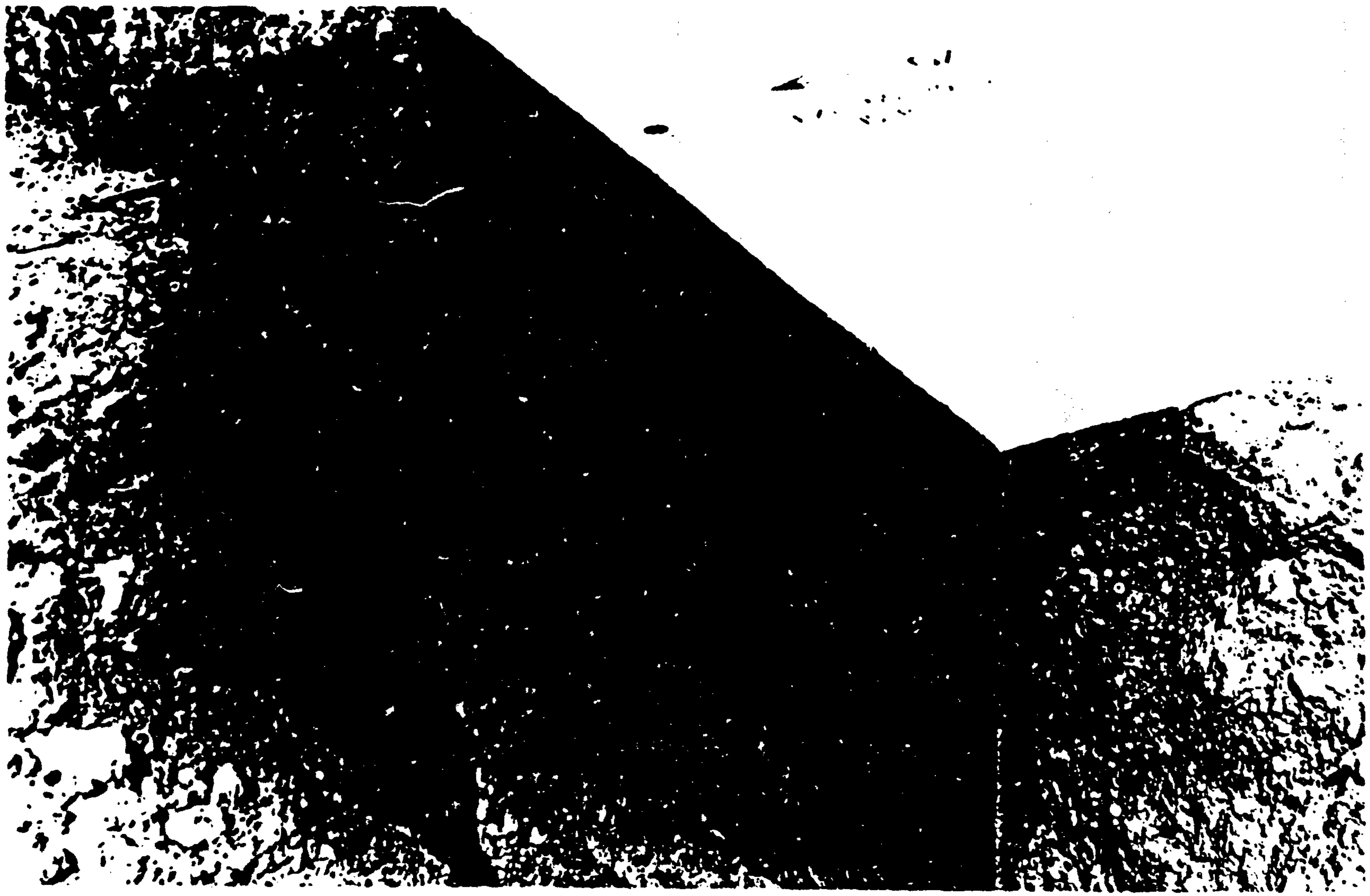


FIGURE 43 -- Post-Blast View of Hospital Shelter at 15 psi Level -- Showing No Cracks

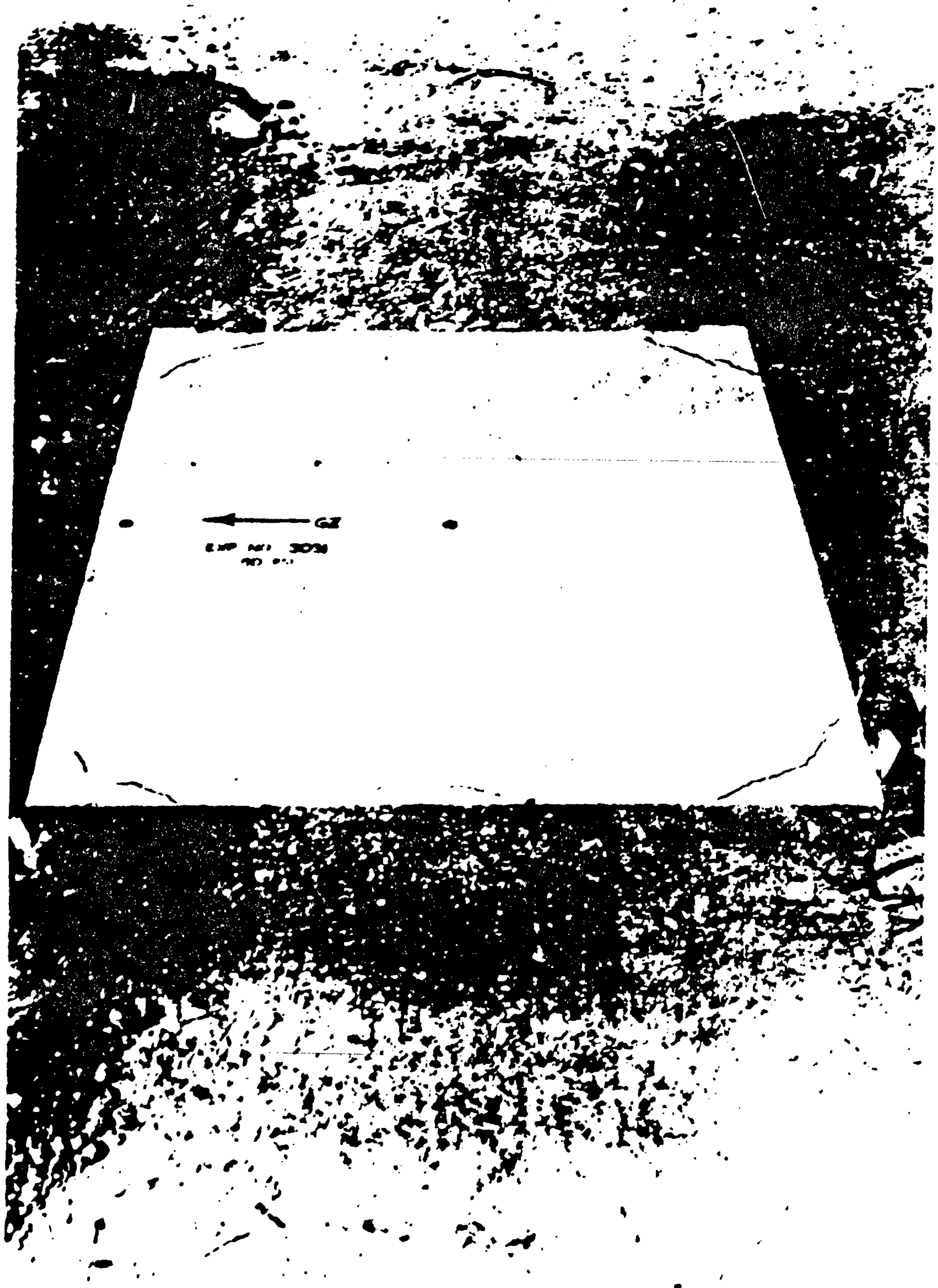


FIGURE 44 -- Post-Blast View of Hospital Shelter at 50 psi Level -- Showing Hairline Cracks on Roof

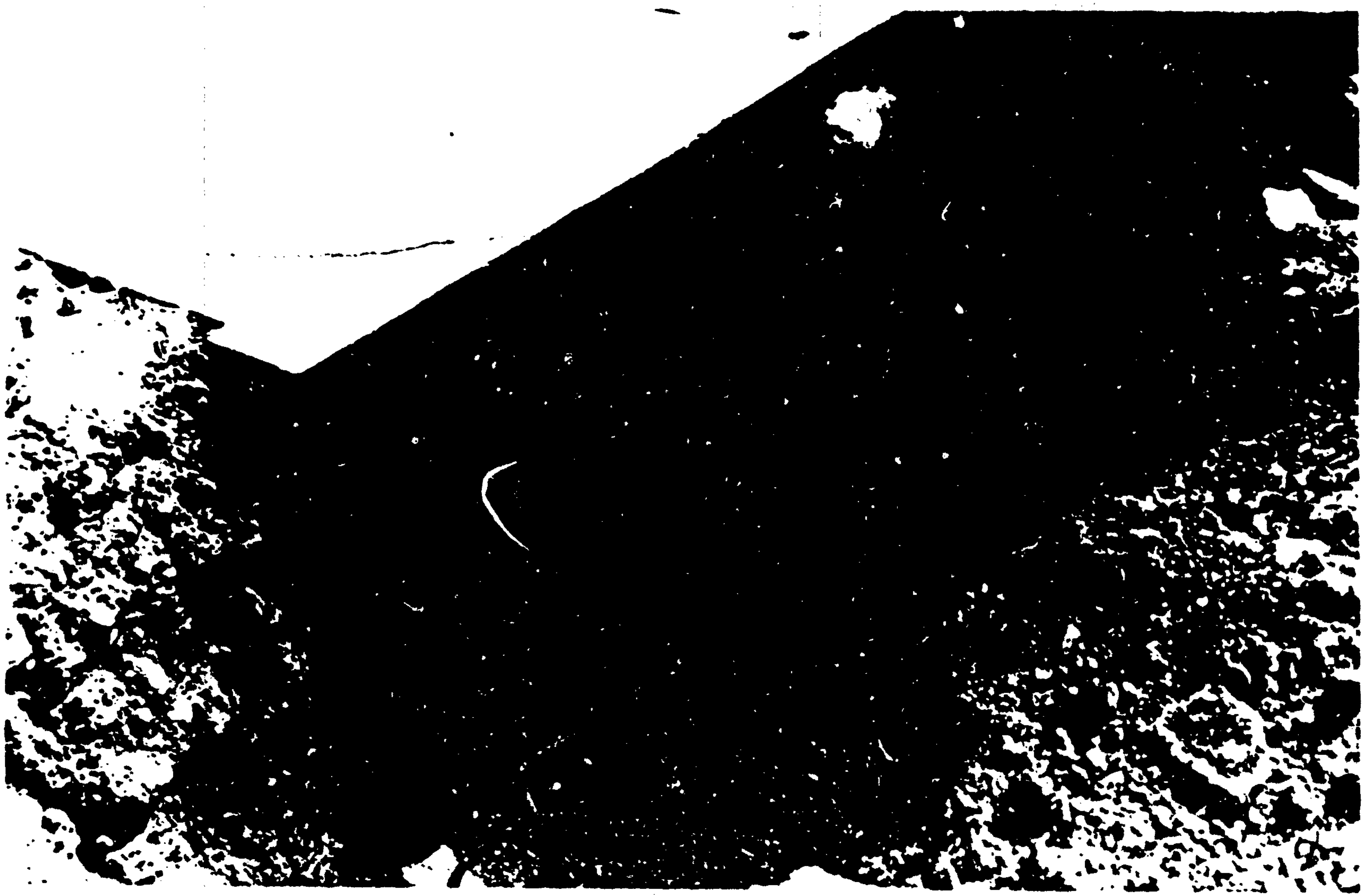


FIGURE 45 -- Post-Blast View of Hospital Shelter at 50 psi Level -- Showing Hairline Cracks

CONCLUSIONS

Conclusions can be drawn in regard to two major issues which were investigated.

- Feasibility of incorporating blast resistant shelters into otherwise typical building projects.
- Adequacy of currently existing shelter design guidance materials.

No conclusion as to the cost implications of a large-scale blast shelter construction program can be drawn from this effort.

Blast shelter construction as an add-on proposition to structures being built for totally different purposes is a viable concept. It is a manageable task which can be accomplished rather simply given adequate financing and proper planning and forethought. Given the significant number of variables over which FEMA has no control, however, it would be extremely difficult to predict precisely when the construction of any particular shelter might be completed. There was a significant increase in anti-nuclear sentiment during the course of this project which might also tend to make the task more difficult.

Existing design guidance is considered inadequate for use in a large scale shelter construction program. The materials used for this project assumed prior knowledge, provided more education than assistance, and were difficult and taxing to use. The information and recommendations which the documents provided were proven to be accurate, though apparently based on highly conservative assumptions.

RECOMMENDATIONS

Any large-scale shelter construction program should be formulated in recognition of the exigencies of the construction industry, financial markets, and the whims of public opinion relative to nuclear issues.

No such national program should be contemplated prior to the preparation of simplified, straightforward design manuals for architects and engineers as well as generalized educational materials for building owners and others involved in the construction process.

APPENDIX A:

PRELIMINARY STRUCTURAL CALCULATIONS -- HOSPITAL SHELTER

TRY 30' SPAN BEAMS WITH 10' SPACINGS   15 PSI OVERPRESSURE LOADING

15 PSI = 15 x 144 = 2160 PSF.

BEAM LOADING = 2160 x 10' = 21600 #/'

$M = 1/8 \times 21600 \times 30.0^2 = 2430'^k$ $d = \sqrt{\frac{2430}{0.579 \times 2}} = 45.8''$ $As = 2430/4.02 \times 46 = 13.14 in^2$

$\rho = 13.14/24 \times 46 = 0.0119$

TRY BEAM 24" x 48" (CHANGES TO 30 x 48)

$Mu = 0.91 \times 0.0119 \times 24 \times 46^2 \times 52000 psi = 2383'^k$ vs 2430   12 # 10's $As = 12 \times 1.27 = 15.24 in^2$

$$q_y\left(\frac{a}{b}\right) = 8[0.0119(1-6.13 \times 0.0119) + 0.0025(1-6.13 \times 0.0025)] 72000 \left(\frac{44}{360}\right)^2 = 116.1 psi$$

$q_y = 116.1 \times \frac{24}{120} = 23.2 psi > 15 PSI$

DIAGONAL TENSION

$$q_{dt} = \left(\frac{1}{2 + \frac{0.0025}{0.0119}}\right) \times (1000 + 2 \times 0.0025 \times 42000)\sqrt{0.0119 \times 3750} \times \left(\frac{44}{360}\right)^2 \times \frac{24}{120} = 10.9 \text{ psi} < 15 \text{ N.G.}$$

INCREASE WEB STEEL 4 # 4 @ 9" = $4 \times 0.20 \times \frac{12}{9} = 1.07 in^2$ $\rho = \frac{1.07}{24 \times 9} = 0.0049$

$$q_{dt} = \left(\frac{1}{2 + \frac{0.0025}{0.0119}}\right) \times (1000 + 2 \times 0.0049 \times 42000)\sqrt{0.0119 \times 3750} \times \left(\frac{44}{360}\right)^2 \times \frac{24}{120} = 12.7 \text{ psi} < 15 \text{ N.G.}$$

WIDEN BEAM TO 30"   4 # 4 @ 6" $\rho_v = \frac{0.80}{30 \times 6} = 0.0044$ $\rho = \frac{15.24}{30 \times 44} = 0.0115$

$$q_{dt} = \left(\frac{1}{2 + \frac{0.0025}{0.0115}}\right) \times (1000 + 2 \times 0.0044 \times 42000)\sqrt{0.0115 \times 3750} \times \left(\frac{44}{360}\right)^2 \times \frac{30}{120} = 15.1 \text{ psi} \simeq 15.0 \text{ psi O.K.}$$

INCREASE STEEL YIELD STRENGTH

$$q_{dt} = \left(\frac{1}{2 + \frac{0.0058}{0.0115}}\right) \times (1000 \times 2 \times 0.0044 \times 7200)\sqrt{0.0115 \times 3750} \times \left(\frac{44}{360}\right)^2 \times \frac{30}{120} = 18.1 \text{ psi}$$

$V = \frac{30}{2} \times 21600 = 324^k$ $v = \frac{324000}{30 \times 44} = 245 psi$ 289 PSIU

PURE SHEAR $d/L = 44/360 = 0.122 < 0.20$

$$q_{dv}\left(\frac{a}{b}\right) = 0.44 \times 3000 \left(\frac{0.122}{1-0.122}\right) = 183.8$$ $q_{dv} = 183.8 \times \frac{30}{120} = 45.9 \text{ psi} > 15.0$ O.K.

NRH - BLAST SHELTER

NEUBAUER - SOHN, CONS. ENGRS   NOV 15.82 1

SLAB SPAN 10'-0"

$M = \frac{1}{10} \times 2160 \times 10.0^2 = 21600\ ^{'\#}/FT.$ $A_s = 21.6/4.02 \times 1.2 \times 6.5 = 0.69\ in^2/FT$

$p = 0.69/12 \times 6.5 = 0.0088 > 0.005$

FLEXURAL YIELD RESISTANCE FOR COMPRESSIVE STEEL USE $0.0088/2 = 0.0044$

INTERIOR SPAN

$q_Y\left(\frac{a}{b}\right) = 8\left(1 - 6.13 \times 0.0088\right)(0.0088 + 0.0088)\,72000\left(\frac{6.5}{120}\right)^2 = 28.1\text{ psi} > 15.0 \quad (a/b = 1)$

END SPAN

$q_Y\left(\frac{a}{b}\right) = 8\left(1 - 6.13 \times 0.0088\right)(0.0088 + 0.0044)\,72000\left(\frac{6.5}{120}\right) = 20.9\text{ PSI}$

PRESHEAR $d/L = 6.5/120 = 0.05 < 0.20$ 7½ SLAB

$q_V\left(\frac{a}{b}\right) = 0.44 \times 3000\left(0.0542/1 - 0.0542\right) = 75.6\text{ psi} < 15.0$ O.K.

DIAGONAL TENSION

$q_t = \left(\frac{1}{2 + \frac{0.0025}{0.0088}}\right)\left(1000 + 2 \times 0.0028 \times 42000\right)\sqrt{0.0088 \times 3000} \times \left(\frac{6.5}{120}\right)^2 \times \frac{12}{12} = 8.2\text{ psi} < 15.0$ N.G.

#3 @3" $A_s = 0.10$ $p = \frac{0.10}{12 \times 3.0} = 0.0028$ $A_s = \frac{2 \times 0.10}{12 \times 3} = 0.0056$

$q_t = \left(\frac{1}{2 + \frac{0.0025}{0.0088}}\right)\left(1000 + 2 \times 0.0056 \times 42000\right)\sqrt{0.0088 \times 3000} \times \left(\frac{6.5}{120}\right)^2 = 9.7\text{ psi} < 15.0$

TRY 9" SLAB d = 7½" $p = 0.63/12 \times 7.5 = 0.0070$

$q_t = \left(\frac{1}{2 + \frac{0.0025}{0.0070}}\right)\left(1000 + 2 \times 0.0056 \times 42000\right)\sqrt{0.0070 \times 3000} \times \left(\frac{7.5}{120}\right)^2 = 11.2\text{ psi}$

TRY HIGHER STEEL STRESS

$q_t = \left(\frac{1}{2 + \frac{0.0025}{0.0070}}\right)\left(1000 + 2 \times 0.0056 \times 72000\right)\sqrt{0.0070 \times 3000} \times \left(\frac{7.5}{120}\right)^2 \times \frac{12}{12} = 13.7\text{ psi} < 15.0$

TRY 4000 psi CONCRETE

$q_t = \left(\frac{1}{2 + \frac{0.0025}{0.007}}\right)\left(1000 + 2 \times 0.0056 \times 72000\right)\sqrt{0.007 \times 4000} \times \left(\frac{7.5}{120}\right)^2 \times \frac{12}{12} = 15.8\text{ psi} > 15.0$ O.K.

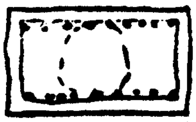
COLUMN

DYNAMIC LOAD = 10'×12"×12" ×15 PSI = 21,600#/' × 30'/2 = 324,000# /BM REACTION

2×324k = 648k × 2 (LOADING FACTOR) = 1296k

TRY COLUMN 30" (TO MATCH BEAM WIDTH)×18"   4% REBAR = 0.040×30×18 = 21.6 in²

18#10 = 18×1.27 = 22.86 in²   p = 22.86/30×18 = 0.0423



0.85×4000×1.25×30×18 + 22.86×72000 = 3941k

0.333×30×5000×16.5² + 11.43×72000×13 = 1892 ik

$\frac{P}{3941} + \frac{0.1 \times 1.5 \times P}{1892} = 1$ P + 0.312 P = 3941   P = 3941/1.312 = 3003k ULT

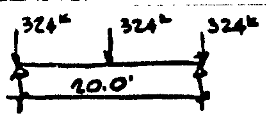
P' = 3003×0.70 = 2102k > 1296k

HOWEVER, IT IS RECOMMENDED THAT SPIRAL TIED COLUMNS BE USED FOR BLAST DESIGN

TRY 18"φ COL / 8#10's

FROM CRSI @ 0.1t   Pu = 743k × 1.2 (DYNAMIC STRESSES) = 892k   CLOSE ENOUGH

GIRDER SPAN 20.0'



−M = 0.150×324×20 = 972 ik   +M = 0.175×324×20 = 1134 ik

$d = \sqrt{\frac{1134}{0.579 \times 1.5}} = 36.1$ USE 80"   (18"×84" BM)

As = 1134/4.02×1.2×80 = 2.94 in²   MIN As = 0.005×18×80 = 7.2 in²

6#10 = 7.62 in²   p = 7.62/18×80 = 0.0053

AV = 0.0025×18×12 = 0.54 in²   2#5 = 0.62 in²

Pv = 0.62/18×12 = 0.0029

PURE SHEAR   d/L = 80/240 = 0.333

$q_v\left(\frac{a}{b}\right) = 0.55 \times 4000 \times \frac{18}{240} = 206.3$ $q_v = \frac{206.3 \times 18}{240} = 15.5$ psi > 15.0 OK

FLEXURE

$q_4\left(\frac{a}{b}\right) = 8(1 - 6.13 \times 0.0053)(0.0053 + 0.0053) \times 72000 \left(\frac{80}{240}\right)^2 = 656.4$

$q_4 = 656.4 \times \frac{18}{120} = 98.5$ psi > 15.0 O.K.   INTERIOR SPAN

END SPAN

$q_4\left(\frac{a}{b}\right) = 8(1 - 6.13 \times 0.0053)(0.0053 + 0.0026) \times 72000 \times \left(\frac{80}{240}\right)^2 = 489.2$

$q_4 = 489.2 \times \frac{18}{240} = 73.4$ psi > 15.0   O.K.

DIAGONAL TENSION

$$q_u = \left(\frac{1}{2+\frac{0.0053}{0.0053}}\right)\left(1000+2\times0.0025\times72000\right)\sqrt{0.0053\times4000}\times\left(\frac{80}{240}\right)^2\times\frac{18}{120} = 302\ \text{psi} > 15.0$$

NATURAL PERIODS

SLAB $T = \frac{1}{850{,}000\sqrt{0.010}}\times\frac{120^2}{7.5} = 0.0226\ \text{sec}$

$\mu = 5$

$t_d/T = 1.6/0.0226 = 70.8 \qquad \frac{p_m}{r_y} = 0.91 \qquad \frac{t_m}{T} = 1.6 \qquad \frac{r}{r_y} = -0.13$

BEAM

$p = 0.0118 \qquad T = \frac{360^2}{425000\sqrt{0.0118}\ 44} = 0.064\ \text{sec}$

$\mu = 3 \qquad \frac{t_d}{T} = \frac{1.6}{0.064} = 25 \qquad \frac{p_m}{r_y} = 0.85 \qquad \frac{t_m}{T} = 1.0 \qquad \frac{r}{r_y} = -0.25$

WALL

$p = 0.0083 \qquad T = \frac{156^2}{450{,}000\sqrt{0.0083}\times10} = 0.059$

$t_d/T = 1.6/0.059 = 27.0 \qquad p_m/r_y = 0.96 \qquad \frac{t_m}{T} = 1.6 \qquad \frac{r}{r_y} = -0.17$

$\mu = 3$

GIRDER

$p = 0.0053 \qquad T = \frac{240^2}{450{,}000\sqrt{0.0053}\times150} = 0.0117$

$\mu = 3$

$t_d/T = 1.6/0.0117 = 136.7 \qquad \frac{p_m}{r_y} = 0.84 \qquad \frac{t_m}{T} = 1.05 \qquad \frac{r}{r_y} = -0.22$

WALL

$M = 1/8\times2160\times13.0^2 = 45.63^{IK} \qquad d = \sqrt{\frac{45.63}{0.579}} = 8.87" \qquad$ 12" wall $d = 10" \qquad A_s = 45.63/4.02\times1.2\times1$

#9 @12" $A_s = 1.0\ \text{in}^2/\text{ft}$

$p = \frac{1.0}{12\times10} = 0.0083 \qquad d/L = \frac{10}{156} = 0.064 < 0.20$

PURE SHEAR

$q_v\left(\frac{a}{b}\right) = 0.46\times4000\sqrt{\frac{0.064}{1-0.064}} = 120.3\ \text{psi} > 15\cdot0.14$

DIAGONAL TENSION

$$q_t\left(\frac{a}{D}\right) = \left(\frac{1}{2 + \frac{0.0025}{0.0083}}\right)\left(1000 + 2 \times 0.0025 \times 72000\right)\sqrt{0.0083 \times 4000} \times \left(\frac{10.5}{156}\right)^2 \times \frac{12}{12} = 15.62 > 15.0$$

FLEXURE

$$q_y\left(\frac{a}{D}\right) = 8\left(1 - 6.13 \times 0.0083\right) \times 0.0083 \times 72000 \times \left(\frac{10.5}{156}\right)^2 = 20.6 \text{ psi} > 15.0$$

RESISTANCE

SLAB, FLEXURAL (q_y) = 20.9 PSI $n = -0.13\, n_y < 0.50$ O.K.
SHEAR (q_v) = 75.6 PSI
DIAG. TEN (q_t) = 37.5 PSI

BEAM, FLEXURAL (q_y) = 25.5 PSI $\frac{n}{n_y} = -0.41$ $n = -0.41\, n_y < 0.50$ O.K.
SHEAR (q_v) = 45.9 PSI
DIAG. TEN (q_{te}) = 18.1 PSI

WALL, FLEXURAL (q_y) = 20.6 PSI $\frac{n}{n_y} = -0.47$ $n = 0.47 < 0.50$ O.K.
SHEAR (q_v) = 120.3 PSI
DIAG. TEN (q_t) = 15.4 PSI

GIRDER, FLEXURAL (q_y) = 73.4 PSI $\frac{n}{n_y} = -0.22$ $n = 0.22\, n_y < 1.0$ O.K.
SHEAR (q_v) = 15.5 PSI
DIAG. TEN (q_t) = 30.2 PSI

NRH - BLAST SHELTER

NEUBAUER & SOHN, CONS. ENGRS.

APPENDIX B:

FINAL STRUCTURAL CALCULATIONS -- HOSPITAL SHELTER

NATIONAL REHABILITATION

HOSPITAL

BLAST SHELTER

CALCULATIONS

AUGUST, 1983

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By: [illegible] | Check | Date: [illegible]
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[illegible]

DESIGN ROOF SLAB $L = 13'$ $P_{so} = 15$ PSI = [illegible]

$f'c = 3000$ PSI $f'dc = 3750$ PSI $f_{dy} = 72000$ [illegible]

ASSUME μ FOR EACH MODE OF FAILURE $\mu = \frac{0.1}{\rho - \rho'}$

1) FLEXURE $\mu = 10$ $\rho \approx \rho' = 0.02$

2) PURE SHEAR $\mu = 1.3$

3) DIAGONAL TENSION $\mu = 3.0$ $\rho_v > 0.0025$
$\mu = 1.5$ $\rho_v < 0.0025$

CALCULATE REQUIRED RESISTANCE FOR EACH MODE $\frac{P_{so}}{(1-\frac{1}{2\mu})}$

FLEXURE $q_f = 15/(1 - \frac{1}{2\times 10}) = 15.6$ PSI

SHEAR $q_v = 15/(1 - \frac{1}{2\times 1.3}) = 24.4$ PSI

DIA TEN. $q_t = 15/(1 - \frac{1}{2\times 3}) = 18$ PSI $\rho_v > 0.0025$

$q_t = 15/(1 - \frac{1}{2\times 1.5}) = 22.5$ PSI $\rho_v < 0.0025$

DEPTH BASED ON SHEAR
$d/L \le 0.2$ & NO INCLINED STEEL $b/a = 1.0$

$q_v = 24.4 = 3000 \frac{d/L}{1 - d/L} \times .44$ -

$24.4 - 24.4\, d/L = 1320\, d/L$ $d/L = .019$

$d = .019 \times 13 \times 12 = 2.9$ IN (MIN)

DEPTH BASED ON FLEXURE

END SPAN $q_f = 7.3(.02 + .01)\times 72000\, (\frac{d}{L})^2 = 15.6$

$d = \sqrt{\frac{15.6}{\ }} = .037$

$d = 13 \times 12 \times .032 = 4.9$ IN

NOTE USE d MIN = 12" $d/L = 3$" FOR ... SPAN

ρ_v MIN = .0025

$$\frac{d}{L} = \frac{12}{13 \times 12} = .08 \qquad \rho^+ = \rho^-$$

$$.08 = \sqrt{\frac{15.8}{7.3 \times (\rho + \frac{\rho}{2}) \times 72000}} \qquad .01 = \frac{15.8}{7.3 \times (\rho + \frac{\rho}{2}) \times 72000}$$

$$15.8 = 5256 \times (\rho + \frac{\rho}{2}) \qquad \rho = .003$$

USE $\rho = \rho$ MIN OF .005

CHECK FOR INTERIOR SPAN

$$15.8 = q_F = 7.3(\rho^+ + \rho^-) \times 72000 \times (\frac{d}{L})^2$$

$$\frac{d}{L} = \sqrt{\frac{15.8}{7.3 \times .01 \times 72000}} = .055$$

$d = .055 \times 12 \times 13 = 8.5" < 12"$ O.K

CHECK FOR END SPAN

$$\frac{d}{L} = \sqrt{\frac{15.8}{7.3 \times .0075 \times 72000}} = .06$$

$d = .06 \times 12 \times 13 = 9.4" < 12"$ O.K

CHECK DIAG TENSION

$$q_T = 3.5\sqrt{3000}\,(.08) = 15.34 < 22.5 \text{ PSI}$$

$$22.5 = [1/(2+1)][1000 + (\rho_v \times 72000)]\sqrt{.005 \times 3000 \times .08^2} \times [1 + 1.5]$$

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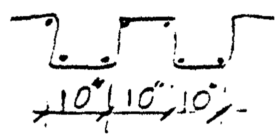
shelter

$225 = 20.5 + 2944 \rho_v$ $\rho_v = \frac{2}{2944} = .0007 < .0025$

USE ρ_v MIN $= .0025$

$.0025 = \frac{A_v}{10 \times 3} = .08$ USE #3 TIES



1) CALCULATE NATURAL PERIOD

$L = 13 \times 12 = 156$ $\rho = .005$ $\rho' = .005$ $d = 12"$

$T = \frac{1 \times 156^2}{638000 \sqrt{.005} \times 12} = .04$ SEC FOR END SPAN

$T = \frac{1 \times 156^2}{850000 \sqrt{.005} \times 12} = .03$ SEC FOR INT. SPANS

SINGLE TRIANGLE REPRESENTATION; CHECK REQUIRED RESISTANCE

$T < T_d$ $.04 < 1.55$

$\frac{T_d}{T} = \frac{.7}{.04} = 17.5$ END SPAN

$\frac{T_d}{T} = \frac{.7}{.03} = 23.3$ INT. SPAN

FLEXURE: $\mu = 10.0$ $\frac{T_d}{T} = 17.5$ $\frac{P_m}{r_y} = 1.01$

FIGURE 4-13 $\frac{T_d}{T} = 23.3$ $\frac{P_m}{r_y} = .99$

q_f REQUIRED $= \frac{15}{.99} = 15.15$ PSI

DIRECT SHEAR $\mu = 1.3$ $\frac{T_d}{T} = 17.5$ $\frac{P_m}{r_y} = .65$

$q_y = \frac{15}{.65} = 23.08$ PSI $\frac{T_d}{T} = 23.3$ " "

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BLAST SHELTER ROOF CONT'D

DIAGONAL TENSION $\mu = 1.5$ $\frac{r_u}{r_y} = .66$

$q_l = \frac{15}{.66} = 22.7$ psi

PROVIDED RESISTANCE

FLEXURE $q_f = 7.3(.005+.005) \times 72000 \times .08^2 = 33.6 > 15$

PURE SHEAR $q_{ps} = .44 \times 3000 \left(\frac{.05}{1-.05}\right) = 114.8$ psi > 15 psi

DIAGONAL TENSION $q_l = [1/(2+1)][1000+28 \times .0025 \times 72000]\sqrt{.005 \times 3000}$
$\times .08^2 \times 25 = 27.8$ psi > 22.7 psi ok

NOTE THE PROVIDED RESISTANCE IN EACH MODE IS GREATER THAN REQUIRED

CHECK REBOUND

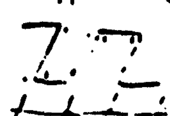
$\mu = 10.0$ $\frac{T}{T} = 17.5$ & 23.3

FIGURE 4-14 $r/r_y = .18$

REQ RE. RESISTANCE $= .18 \times 33.6 = 6.05$ psi

PROVIDED REBOUND RESISTANCE $= 7.3 \times .005 \times 72000 \times .08^2 = [illegible]$

SLAB SUMMARY USE $T = 15"$ $d = 12"$ #7 @ 10" o.c. T & B

#3  @ 3" o/c

10" 10"

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[illegible] SHELTER BEAM

BEAM DESIGN $L = 30'-5"$ $f'c = 3000$ PSI, $f_{dc} = 3750$ PSI

ASSUME $\mu = 10$ FOR FLEXURE $f_{dy} = 72000$ PSI

PURE SHEAR: $\mu = 1.3$ FLEXURE $\mu = 10$

DIAGONAL TENSION $\mu = 3.0$ $p_v > 0.0025$

$\mu = 1.5$ $p_v < 0.0025$

SLAB DEAD LOAD $= \frac{150 \times 14}{1728}\left(1 - \frac{1}{6}\right) = 1.$ PSI FLEXURE

$\frac{150 \times 14}{1728}\left(1 - \frac{1}{2.6}\right) = .748$ PSI PURE SHEAR

DIA. TENSION $\begin{cases} \frac{150 \times 14}{1728}\left(1 - \frac{1}{6}\right) = 1.0 \text{ PSI } p_v > .0025 \\ \quad \left(1 - \frac{1}{3}\right) = .814 \text{ PSI } p_v < .0025 \end{cases}$

$q_F = \frac{16}{1 - \frac{1}{6}} = 19.2$ PSI

$q_V = \frac{15.75}{1 - \frac{1}{2.6}} = 25.5$ PSI

$q_T = \frac{16}{1 - \frac{1}{6}} = 19.2$ PSI $p_v > 0.0025$

$q_T = \frac{15.81}{1 - \frac{1}{3}} = 23.7$ PSI $p_v < .0025$

DEPTH BASED ON SHEAR

$q_V = 25.5 \times \frac{13}{3} = \frac{d/L \times .44 \times 3000}{1 - d/L} = 110.5 - 110.5\, d/L = 1320\, d/L$

$d/L = .09$ $110.5 - 110.5\, d = 1320\, d/L$

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CHECK BEAM DEPTH FOR q_E & q_T

$$19.2\frac{13}{3} = 7.3\times.02\times72000\times\left(\frac{d}{L}\right)^2$$

$$\sqrt{\frac{83.2}{7.3\times.02\times72000}} = \frac{d}{L} = .09$$

DEPTH BASED ON DIAGONAL TENSION

ASSUME $\rho = .005$

$q_T = 3.5\sqrt{3000}(.09)\times\frac{3}{13} = 3.98$ PSI < 23.7 NG

TRY $\rho_V < .0025$ q_T REQUIRED $= 23.7$ PSI

$$23.7 = \left[1/\left(2+\frac{.005}{.02}\right)\right]\left[1000+2\rho_V\times72000\right]\times\sqrt{.02\times3000}\times(.09)^2\times\frac{3}{13}$$

$$23.7 = (.44)(1000+\rho_V\times144000)\times7.75\times.01\times.23$$

$23.7 = 7.84 + 1129.4\rho_V$ $\rho_V = .014 > .0025$

$\therefore \rho_V > .0025$ q_T REQUIRED $= 19.2$ PSI

$$19.2 = \left[1/\left(2+\frac{.005}{.02}\right)\right]\left[1000+144000\rho_V\right]\times\sqrt{.02\times3000}\times.09^2\times\frac{3}{13}$$

$19.2 = 7.84 + 1129.4\ \rho_V$ $\rho_V = .0101$

$\rho_V = \frac{A_V}{b's}$ $A_V = .0101\times36\times6 = 2.18\ IN^2$

$\boxed{4}$ #7 @ 6" o/c FOR 12'-0"

WEB REINFORCING INCREASING

TRY $d = 48$" $d/L = \frac{3.56}{30.67} = .1167$

$$19.2 = \left[1/\left(2+\frac{.005}{.02}\right)\right](1000+144000\rho_V)\times\sqrt{.02\times3000}\times.1167^2\times\frac{3}{13}$$

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Check design cont'd

$19.2 = 10.7 + 1542.4\ \rho_v \qquad \rho_v = \frac{8.5}{1542.4} = .0055$

$\rho_v = \frac{A_v}{b's} \qquad A_v = .0055 \times 36 \times 6 = 1.19\ in^2$

. USE #5 @ 6" o/c

Note adjust ρ for $d = 43''$

f_p REQUIRED = 19.2 psi $\quad d/L = .1167$

$.1167 = \sqrt{\frac{19.2 \times 7\frac{2}{3}}{7.3 \times \rho \times 72000}} \qquad .0136 = \frac{82.3}{525600\rho} =$

$\rho = .0115$

$A_s = .0115 \times 36 \times 42 = 17.3\ in^2$

USE 12 #11 Bott $\qquad \rho = .0124$

$A_s' = .005 \times 36 \times 42 = 7.56$

Note see further calcs $\rho = .02$

USE 6 #10 Top

DYNAMIC ANALYSIS OF BEAM

$L = 30.7 \times 12 = 368\ in \qquad \rho = 0.0115$

$T = \frac{368^2}{425000 \times \sqrt{.0124} \times 42} = .0681'$ SEC

REBOUND $\qquad T_s = .7$ SEC $\qquad \frac{T}{T} = \frac{.7}{.0681} = 10.3$

$\mu = \frac{.1}{.0115 - .005} = 15.3 > 10$ USE 10

$r/r_y = .225 \qquad f_F = 7.3 \times .0124 \times 72000 (.1167)^2 \times \frac{3}{13} = 20.4$ psi

$r = -.225 \times 20.4 = 4.6$ psi

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BEAM DESIGN CONT'D

PROVIDED REBOUND RESISTANCE.

$r = 7.3 \times .005 \times 72000 (.1167)^2 \times \frac{3}{13} = 8.26$ PSI > 4.6 PSI OK

CHECK RESISTANCE IN EACH MODE

REQUIRED RESISTANCE V.S. PROVIDED

FLEXURE: $\mu = 3.0$ $\quad \frac{t}{T} = 103$ $\quad$ FIG 4-13

$\frac{P_m}{r_f} = .85$ $\quad r_f = \frac{16}{.85} = 18.8$ PSI

PROVIDED $r_f = 20.4$ PSI > 18.8 OK

PURE SHEAR

$\mu = 1.3$ $\quad \frac{T}{10} = 10.7$

$\frac{P_m}{r_v} = .65$

$r_v = \frac{15.8}{.65} = 24.3$ PSI

PROVIDED $r_v = .55 \times 3000 \times .1167 \times \frac{3}{13} = 44.4$ PSI > 24.3 PSI OK

DIAGONAL TENSION $\quad \frac{t}{T} = 10.3$ $\quad \mu = 3.0$ $\quad r_T = 18.8$ PSI

r_T PROVIDED $= \left[1/(2 + \frac{.005}{.0164})\right] \times (1000 + 44000 \times .0057) \times \sqrt{.0164 \ldots}$

$\times .1167^2 \times \frac{3}{13} = 14.8$ PSI < 18.8 PSI N.G.

REDESIGN BM TRY BM 2'-0" × 4'-0"

$d/L = \frac{46}{30 \times 12} = .128$ $\quad$ DETERMINE ρ

$\frac{19.2 \times 13}{2} = 7.3 \times \rho \times 72000 \times .128$ $\quad \therefore 8611.4 \rho = 124.8$

$\rho = .0145$

USE $\bar{\rho} = .015$

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BEAM REDESIGN

PURE SHEAR BY INSPECTION O.K. $\rho_v' = .005$

σ_T REQUIRED DIA TENSION = 19.2 PSI $\rho = .015$ $\frac{d}{L} = .128$

$$19.2 = \left[1/(2 + \frac{.005}{.015})\right]\left[1000 + 144000\rho_v\right] \times \sqrt{.015 \times 3000} \times .128^2 \times \frac{2}{13}$$

$19.2 = 7.2 + 1043.5\rho_v$ $\rho_v = .0115$

$A_V = .0115 \times 24 \times 6 = 1.65$

TRY BM 4'-0 x 4'-0"

$\frac{19.2 \times 13}{4} = 7.3 \times \rho \times 72000 \times .128^2$ $\rho = .0072$

USE $\rho = .0075$

$$19.2 = \left[1/(2 + \frac{.005}{.0075})\right](1000 + 144000\rho_v) \times \sqrt{.0075 \times 3000} \times .125^2 \times \frac{4}{13}$$

$19.2 = 8.96 + 1291.3\rho_v$ $\rho_v = .0069$

$A_V = .0069 \times 48 \times 6 = 2.0 \text{ IN}^2$

TRY BM. 36 x 44 W/ INCREASED ρ_v

$$19.2 = \left[1/(2 + \frac{.005}{.024})\right] \times (1000 + 144000\rho_v) \times \sqrt{.024 \times 3000} \times .1167^2 \times \frac{3}{13}$$

$19.2 = 8 + 1148.5\rho_v$ $\rho_v = .0098$

$A_V = .0098 \times 36 \times 6 = 2.11 \text{ IN}^2$ UNREALISTIC

USE BM 36 x 45 WITH $\rho = .02$

$$\sigma_T = \left[1/(2 + \frac{.005}{.02})\right] \times (1000 + 144000 \times .0057) \times \sqrt{.02 \times 3000} \times .1167^2 \times \frac{3}{13}$$

$\sigma_T = 19.7$ PSI > 19.2 PSI O.K.

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BM REDESIGN CONT.

REPEAT DYNAMIC ANALYSIS $L = 368$ IN $\rho = .02$

$$T = \frac{368^2}{425000\sqrt{.02 \times 42}} = .0536 \qquad t_s = .7 \text{ SEC} \quad \text{FIG. 2-14(a)}$$

$$\frac{t}{T} = \frac{.7}{.0536} = 13.05 \qquad \mu = \frac{.1}{.02-.005} = 6.67 \qquad \frac{P_m}{r_f} = .96$$

FLEXURE

$$r_f \text{ REQUIRED} = \frac{16}{.96} = 16.3 \text{ PSI}$$

$$r_f \text{ PROVIDED} = 7.3 \times .02 \times 72000 \times .1167^2 \times \frac{3}{13} = 33 \text{ PSI} > 16.3 \text{ o.k}$$

PURE SHEAR $\mu = 1.3$ $\frac{P_m}{r_v} = .65$

$$r_v = \frac{15.8}{.65} = 24.3 \text{ PSI REQUIRED}$$

$$r_v \text{ PROVIDED} = .44 \times 3000 \left(\frac{.1167}{1-.1167}\right) \times \frac{3}{13} = 40.2 \text{ PSI} > 24.3 \text{ o.k}$$

DIA. TENSION $\mu = 3.0$ $\frac{t}{T} = 13.6$ $\frac{P_m}{r_T} = .85$

$$r_T \text{ REQUIRED} = \frac{16}{.85} = 18.8 \text{ PSI}$$

r_T PROVIDED = 19.7 PSI SEE SHT 9

19.7 PSI > 18.8 o.k

CHECK REBOUND $t_s = .7$ SEC. $\frac{t}{T} = 13.05$ $\mu = 6.61$

$$\frac{r}{r_y} = .21 \qquad \frac{\rho'}{\rho} = \frac{.005}{.02} = .25 > .21 \text{ o.k}$$

BEAM SUMMARY

BM : 36" WIDE × 45" DEEP

BOTT STEEL = 20 # 11 IN TWO LAYERS

TOP STEEL = 6 # 10 TOP

5 @ 6" O/C

USE d = 40" FLEX

D = 42" SHEAR

d = ... FOR SR ... CLEAR SPAN = 26.6" $d = .1167 \times ... = 39.9$

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WALL DESIGN

DESIGN WALL FOR $L = 14'-6"$ $P_{so} = 15$ psi $= B_0$

$f_{dy} = 72000$ $f'_{dc} = 3750$ $f'_c = 3000$ p.

REQUIRED RESISTANCE $= q_F = 15.6$ psi

$q_Y = 24.4$ psi

$q_T = 18$ psi $P_V > .0025$

$q_T = 22.5$ psi $P_V < .0025$

DEPTH BASED ON SHEAR

$q_Y = .55 f'_c \frac{d}{L}$ $\frac{d}{L} = \frac{24.4}{.55 \times 3000} = .015$

$d = .015 \times 14.5 \times 12 = 2.6$ in

DEPTH BASED ON FLEXURE $P = .005$ $P' = .0025$

$$\frac{d}{L} = \sqrt{\frac{15.6}{(73)(.005)(72000)}} = .0775$$

$d = .0775 \times 14.5 \times 12 = 13.5$

TRY $P = .01$ $\frac{d}{L} = \sqrt{\frac{15.6}{7.3 \times .01 \times 72000}} = .0548$

$d = .0548 \times 14.5 \times 12 = 9.5"$ USE $T = 12"$ $d = 10"$

CHECK DIA. TENSION $q_T = 3.5\sqrt{3000} \times .0575 = 11$ psi

$$q_T = \left(\frac{1}{2 + \frac{0.0025}{.01}}\right) \times (1000) \times \sqrt{.01 \times 3000} \times .0575^2 = 8.\ \text{psi}$$

USE WEB STEEL

WALL DESIGN CONT'D

$\rho_v < .0025 \quad \mu = 1.5 \quad \nu_T = 22.5$ PSI $\quad \rho = .0025$

$\rho = .01$

$22.5 = \left[1/\left(2 + \frac{.0025}{.01}\right)\right](1000 \times 144000\ \rho_v)\sqrt{.01 \times 3000} \times .0575$

$22.5 = (.444)(1000 \times 144000\ \rho_v)(5.4) \times .0033$

$22.5 - 7.9 = 1139\ \rho_v \qquad \rho_v = .0128 > .0025$

$\mu = 3 \qquad \nu_T$ REQ $= 18$ PSI

$18 = \left[1/\left(2 + \frac{.0025}{.01}\right)\right](1000 \times 144000\ \rho_v) \times \sqrt{.01 \times 3000} \times .0575$

$18 = 7.9 + 1139\ \rho_v \qquad \rho_v = \frac{10.1}{1139} = .0089$

TRY $d = 12"$ $T = 14"$

$\frac{d}{L} = \frac{12}{12 \times 14.5} = .069$

$\rho = \frac{15.8}{7.3 \times 72000 \times .069^2} = .0063$ USE ρ of $.0065$

DEPTH FOR PURE SHEAR BY INSP. O.K

ASSUME $\rho_v > .0025 \qquad \mu = 3 \qquad \nu_T$ REQ $= 18$ PSI

$18 = \left[1/\left(2 + \frac{.0025}{.0065}\right)\right] \times (1000 \times 144000\ \rho_v) \times \sqrt{.0065 \times 3000} \times .069^2$

$18 = (.419) \times (1000 \times 144000\ \rho_v) \times .021$

$18 = 8.8 + 1268\ \rho_v \qquad \rho_v = \frac{9.2}{1268} = .0073$

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WALL DESIGN DYNAMIC ANALYSIS

Natural period $L = 14.5 \times 12 = 174$ $d = 12''$

$T = \dfrac{174^2}{4 \cdot 5000\sqrt{.0065} \times 12} = .0736$ SEC

$t = .7$ SEC FIG. 2-14(a) $\dfrac{t}{T} = 9.5$

CHECK REBOUND $\dfrac{T}{t} = 9.5$ $\mu = \dfrac{.1}{.0065-.0025} = 25 > 10$

$r/r_y = .225$ $\dfrac{\rho'}{\rho} = \dfrac{.0025}{.0065} = .38 > .225$ ok USE 10

CHECK RESISTANCE IN EACH MODE

FLEXURE $\mu = 10$ $\dfrac{t}{T} = 9.5$ $\dfrac{P_m}{q_f} = 1.05$

$q_{f_{REQ}} = \dfrac{15}{1.05} = 14.2 < 15.8$ ok

PURE SHEAR $\mu = 1.3$ $\dfrac{t}{T} = 9.5$ $\dfrac{P_m}{q_v} = .66$

$q_{v_{REQ}} = \dfrac{15}{.66} = 22.7$ BY INSP. ok

DIA. TEN. $\mu = 3$ $\dfrac{t}{T} = 9.5$ $\dfrac{P_m}{q_T} = .85$

$q_{T_{REQ}} = \dfrac{15}{.85} = 17.6$ > 18 o.k

WALL SUMMARY $t = 14'$ $d = 12''$

$A_s = .0065 \times 12 \times 12 = .936$ USE #8 @ 10" o/c I.F.
$A_s' = .0025 \times 12 \times 12 = .36$ USE #5 @ 10" o/c O.F.
$A_v = .0073 \times 10 \times 3 = .219$ USE #4 @ 3" c/c

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WALL CHECK FOR AXIAL LOAD

CONSIDERER 1' STRIP OF WALL $\rho_T = .0065 + .0025$

$\rho = .009$

$P_u = (.85 \times 3750 + .009 \times 72000) \times 10$

$P_u = 38355$ #

$P = 1 \times (6.5 \times 12) \times 16 = 1248$ #

$\frac{P}{P_u} = \frac{1248}{38355} = .0325$ $\frac{M}{M_u} = 1$ $\frac{\rho f_y}{f'c} = .156$

BY INSPECTION WALL OK

CHECK WALL FOR BEAM REACTION

TRIB AREA $= 15 \times 13 \times 12^2 = 28080$

ASSUME $\mu = 1.0$ DESIGN LOAD $= \frac{2 \times 16 \times 28080}{1000} \approx 900^k$

ASSUME 3 FT OF WALL UNDER BEAM.

$P_u = 36 \times 38.4 = 1382$ K

$\frac{P}{P_u} = \frac{900}{1382} = .65$ $\frac{M}{M_u} = 1.0$ $\frac{\rho f_y}{f'c} = .156$

FROM FIGURE 3-22 $\frac{P}{P_u} = .7 > .65$ o.k

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COL DESIGN

COL. LOAD = 900 K μ = 1.0

ASSUME ρ = .02 NOTE: COL. BRACED BY ...

$900 = (.85 \times 3.75 + .02 \times 72) A'c$

$A'c = \frac{900}{4.63} = 194 \text{ IN}^2 = \frac{\pi d_c^2}{4} =$

$d_c^2 = \frac{194 \times 4}{3.14} = 247 \qquad d_c = 15.72 \text{ IN}$

TRY ρ = .01 $\qquad A'c = \frac{900}{3.91} = 230 \text{ IN}^2$

$d_c^2 = \frac{230 \times 4}{3.14} = 292 \qquad d_c = 17.$

NOTE COL SIZE USED 22×22

USE $d_c = 18''$

ASSUME 2" OF COVER $d_T = 22''$

SPIRAL STEEL REQ'D

$\rho_s = .45 (A_g/A_c - 1) f'c/f_y$

$A_g = \frac{\pi d^2}{4} = \frac{3.14 \times 22^2}{4} = 380$

$\rho_s = .45 \left(\frac{380}{254} - 1\right) \times 3/72 = .01$

OR $\rho_s = .12 \, f'c/f_y = .12 \times 3/72 = .01$

$\rho_s = \frac{4 A_s}{b \, d_c} = .01 \qquad$ TRY #3 BAR $A_s = .11 \text{ IN}^2$

$b = \frac{4 \times .11}{.01 \times 18} = 2.44 \text{ IN} \qquad$ SAY $2\tfrac{1}{4}$"

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COL DESIGN CONT'D

COMPRESSION STEEL

$P_T = .01 = \frac{AS}{3.14 \times 18^2/4}$ $AS = 2.54$ IN^2

NOTE STEEL AREA USED FOR BLDG IS GREATER THAN REQ. FOR BLAST. USE SAME STEEL REQ. FOR BLDG WITH SPIRAL TIES #3

#3 @ 5"

DESIGN STRIP ON THE SIDES OF DOOR TO CARRY DOOR

USE WIDTH = 3'-0" L = 14'-0

μ FLEXURE = 10 DIA. TEN. $\mu = 3.0$ $P_V > .0025$
μ PURE SHEAR = 1.3 $\mu = 1.5$ $P < .0025$

$q_F = 15.6$ PSI $q_V = 25.5$ PSI

$q_F = 19.2$ PSI $P_V > .0025$ $q_V = 23.7$ PSI $P_V < .0025$

DEPTH BASED ON SHEAR

$q_V = 25.5 \times \frac{3}{1} = \frac{d/L \times .04 \times 3000}{1 - d/L}$

$1320\, d/L = 76.5 - 76.5\, d/L$ $d/L = \frac{76.5}{1396} = .055$

ASSUME $P = .02$

DEPTH REQUIRED FOR FLEXURE

$15.6 \times 3 = 7.3 \times .02 \times 72000 \left(\frac{d}{L}\right)^2$ $\frac{d}{L} = \sqrt{\frac{47.4}{7.3 \times .02 \times 72000}} = .067$

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DESIGN WALL TO CARRY BLAST LOAD

DEPTH FOR DIA. TENSION

ASSUME μ = 3 ρ > .0025 θ_T = 19.2 PSI d/L = 2.07

$19.2\left[1/\left(2+\frac{.005}{2}\right)\right]\left[1000+144000\rho_Y\right]\times\sqrt{.02\times3000\times.07} < \frac{1}{4}$

$192 = (.44)(1000+144000\rho_Y)(.013)$

$19.2 = 5.56 + 801\rho_Y$ $19.2-5.56 = 801\rho_Y$

$\rho_Y = \frac{13.6}{801} = .02 < .0025$ $.0025\times3\times12 = .09$

NOTE WEB REINF USED IN WALL IS #4 @ 3' C/C

DYNAMIC ANALYSIS $L = 14\times12 = 168$ IN $\rho = .02$

$T = \frac{168^2}{425000\times\sqrt{.02}\times12} = .039$ $t_s = .7$ SEC FIG 2-4

$\frac{t}{T} = \frac{.7}{.039} = 17.9$

$\mu = \frac{.1}{.02-.005} = 6.66$

CHECK REBOUND $r/r_y = .18$ FIG. 4-14

$\frac{\rho'}{\rho} = \frac{.005}{.02} = .25 > .18$ O.K

CHECK RESISTANCE FOR EACH MODE

FLEXURE $\mu = 6.66$ $\frac{t}{T} = 17.9$ $\frac{P_m}{\theta_F} = .97$

$\theta = \frac{15}{.97} = 15.46 < 15.8$ O.K

PURE SHEAR $\mu = 1.3$ $\frac{t}{T} = 17.9$ $\frac{P_m}{\theta_Y} = .65$

$\theta_Y = \frac{15}{.65} = 23$ PSI < 25.5 O.K

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DESIGN WALL STEM TO CARRY WORK LD

DIR. TEN. $\mu = 3$ $\frac{I}{T} = 17.9$ $\frac{P_{m}}{\phi f} = 2.55$

$f_r = \frac{15}{.85} = 17.6 < 18$ O.K

NOTE RESISTANCE PROVIDED IN EACH MODE IS GREATER THAN RESISTANCE REQUIRED

SUMMARY $\frac{d}{L} = .07$ $d = 12"$ $T = 14"$

$As = .02 \times 12 \times 12 = 2.88$ USE #11

$A's = .005 \times 12 \times 12 = .72$ in^2 USE #6

NEG STEEL SAME AS [illegible]

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